CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

Each year about 285 million tires are discarded in the United States. Scrap tires are visually offensive, a health and fire hazard, and a part of the solid waste management problem. Legislation by the States and by the Federal government have attempted to regulate the transportation and storage of scrap tires and encourage the development of alternative uses (1). During 1991, this problem assumed greater importance due to provisions in the *Intermodal Surface Transportation Efficiency Act (ISTEA --91)*. Section 1038 of the ISTEA-91 mandated the use of Crumb Rubber Modifiers (CRM) in 5 percent of the asphalt pavements placed in 1994 using the Federal-aid and increasing by 5 percent per year, to 20 percent in 1997 and thereafter. Section 1038 also indicated that the penalty for failure to comply with the mandate would be the loss of equivalent percentage of Federal-aid received, excluding the Interstate completion funds (2).

This mandate was put forth based on the information submitted by the U. S. Department of Transportation (USDOT) and the Environmental Protection agency (EPA) in their report to the U. S. Congress. This report indicated that the use of CRM in asphalt mixes would be a feasible task and would not require any waiver provisions (1). Blending crumb rubber with asphalt was reported to increase the viscosity of the resulting blend. This was said to make the mix more pliable and flexible at low temperatures while remaining stiffer and less plastic at high temperatures. This binder and/or mix modification was reported (3) to impart improved rutting, fatigue, and low temperature cracking resistance to the mixes.

However, the degree of improvement and hence the cost effectiveness of using rubber in asphalt mixes has not been firmly established. One would expect that, if the benefits are documented, the asphalt layer thickness can be reduced and/or pavement design lives could be extended. The State DOT"s faced problems with the use of tire rubber in asphalt mixes because :

1. Very limited information (3) was available on the effectiveness of CRM in improving the pavement performance and most of this information come from the asphalt rubber industry,
2. The addition of rubber increased the cost of the mix by 50- 100% (1), and
3. The penalty for non-compliance of with the ISTEA mandate was the loss of Federal-aid (2),

To address these issues, many State DOT's began evaluating the tire rubber or the Crumb Rubber Modifier (CRM) through laboratory and field studies. During 1993, the Arkansas State Highway and Transportation Department (AHTD) and the Mack-Blackwell National Rural Transportation Research Center (MBTC) at the University of Arkansas, Fayetteville, sponsored the study "TRC 9404 -- Effect of Tire Rubber on Asphalt Mixes." The main objective of this study was to develop an understanding of the behavior of asphalt concrete mixes when modified with CRM. This research project was to focus on the performance related properties of a CRM mix that was to be placed as an overlay on Interstate-40.

The field contractor charged with the construction of the overlays faced considerable difficulties in getting CRM designs meeting the AHTD mix specifications. This delayed the overlay construction by almost a year. During this period, aggregates, asphalt and crumb rubber were procured from the contractor to evaluate CRM mixes in the laboratory. The laboratory studies began on a modest scale of designing CRM mixes (dry process) using the Marshall method. The scope of the study was eventually extended to better understand the behavior of CRM mixes produced using the asphalt-rubber blends. The following studies were undertaken under the extended scope of this project during the delay period:

1. Design of CRM mixes for the "DRY" and "WET" processes using Marshall Method
2. Evaluation of rutting, resilient and tensile characteristics of mixes prepared at their respective Optimum Asphalt Content (OAC)

3. Examination of the effect of asphalt-rubber reaction time on the rheological properties of the modified binder.

4. Determination of the performance grade (PG) of rubber modified asphalt binders using Superpave binder testing instrumentation.

5. Design of CRM & unmodified mixes using Superpave volumetric design method

As the research project neared its completion, amendments were made to the 1995 National Highway Appropriation Bill by the U. S. Congress. The ISTEA mandate pertaining to the use of CRM in asphalt mixes was waived thus giving the State DOT's an option to use the CRM if they desired (4). This report presents the results from this three year study dedicated to examining the effect of CRM on asphalt mixes. Through a wide range of side studies, recommendations pertaining to the use of CRM by the AHTD have been developed.

CHAPTER 2

CRM TECHNOLOGY DEVELOPMENT

Tire rubber has been used in asphalt mixes since the late 60's. With a lot of research being done in this field, there are many terminologies associated with tire rubber modified asphalt concrete mixes. Some of the commonly used terminologies are Crumb Rubber Modifier (CRM), asphalt-rubber, rubber modified asphalt mixes (coarse CRM & fine CRM mixes), rubberized asphalt etc. These terms refer to uses of rubber in asphalt mixes that are different in their mix composition, method of production or preparation and in their physical and structural properties. As a result, the considerations in using the above mentioned materials will be different. This necessitates the need to clearly define the terminologies associated with the rubber modified binders and mixes.

Crumb Rubber Modifier (CRM) is a general term used to identify a group of concepts that incorporate scrap tire rubber into asphalt paving materials. The terminologies associated with these CRM mixes are based on the percentage composition of CRM and asphalt and the mix production process (1,5).

2.1 TERMINOLOGIES ASSOCIATED WITH CRM TECHNOLOGY

Tire Rubber can be introduced into asphalt mixes by either reacting crumb rubber with asphalt at temperatures sufficient to cause physical and chemical changes that result in a modified binder or by blending the CRM with hot aggregates before mixing the same with asphalt to produce a rubber modified mix. The first process of blending asphalt and rubber is known as the "wet," process and the process that first mixes rubber with the aggregates is known as the "dry" process (1,2). Similarly, there are several terminologies associated with the CRM mix production. The McDonald process and Continuous blending technology is used in the context of producing the CRM mixes by the wet process. The PlusRide, Generic and Chunk-rubber technologies are associated with the preparation of CRM mixes by dry process. Figure 2-1 shows the technology associated with the use of CRM in asphalt mixes.

Material	**Process**	**Technology**	**Product**
		McDonald	
	WET		**A-R Binder**
		Continuous Blending	
CRM			
		PlusRide	
	DRY	**Generic**	**RUMAC Mix**
		Chunk Rubber	

Figure 2-1 Terminologies Associated With The Use of CRM In Asphalt Mixes[1]

2.1.1 Asphalt-Rubber

Asphalt -Rubber is a term used to indicate an asphalt cement modified with crumb rubber modifier (1,2). Schuler Et. al (6) define Asphalt-Rubber as a modified binder formulated by the physical and chemical bonding of asphalt cement and ground tire rubber at elevated temperatures. The ASTM specifies (2) a minimum of 15 percent rubber by weight of the total blend to achieve a binder with modified properties. Even though the FHWA definition does not specify the range of rubber to be used to obtain a modified blend (1), ground tire rubber ranging from 18 to 26 percent have been used (6) in the FHWA research projects. Green and Tolonen (7) define asphalt-rubber as an equal blend of rubber and asphalt whose response is primarily rubber-like although those responses are modified by the presence of asphalt. The mixes prepared using Asphalt-Rubber are referred to as A-R Mixes.

2.1.2 Rubberized Asphalt

Green and Tolonen (7) define Rubberized-Asphalt as a mixture of rubber in asphalt whose response is primarily asphalt like, although the responses are modified by the presence of rubber. An example of rubberized asphalt is a blend containing 5 percent natural latex rubber.

2.1.3 Rubber Modified Asphalt Mixes

These are basically dense and open graded asphalt concrete mixes to which ground tire rubber is added as a part of the aggregate component. The percentage of rubber used in these mixes varies from 1 to 3 percent by total weight of the mix. The mixes are not considered to be asphalt-rubber since rubber is not blended with the asphalt cement prior to the mixing with the mineral aggregates. These dense and open graded mixes which are produced by first mixing CRM and mineral aggregates followed with an intimate mixing with asphalt cement are referred to as "asphalt concrete rubber filled" and "friction course rubber filled" mixes or Rubber Modified Asphalt Concrete Mixes (RUMAC) (6). The use of CRM in asphalt mixes has been promoted as a means to both improve the performance of asphalt mixes and benefit the environment. Heitzman (1) indicates that the environmental benefit is the use of a material that otherwise would require space in a landfill.

2.1.4 PlusRide Mixes

These are dry-process mixes wherein the CRM, which is primarily used as a rubber aggregate, is incorporated into aggregates with gap gradation prior to mixing with the asphalt cement (1,2). Figure 2-2 shows the typical aggregate gap gradation adopted in PlusRide mixes (8). The finished product from the PlusRide Technology is referred to as "Coarse CRM

Modified Hot Mix Asphalt Concrete Mix."

2.1.5 Generic Dry or TAK Mixes

These are dry-process mixes in which the gradation of CRM is adjusted to suit the aggregate gradation. Unlike the PlusRide mixes, the gradation of CRM is a two component system in which the fine crumb rubber is believed to interact with the asphalt cement while the coarse crumb rubber performs as an elastic aggregate in the Hot Mix Asphalt Concrete (HMAC) mixes (1,2). Figure 2 -3 shows the typical gradation adopted for the TAK/Generic Mixes. The finished product from the Generic Dry mixes is also referred to as "Fine Crumb Rubber Modified Hot Mix Asphalt Concrete". Heitzman (1) indicates in Figure 2-1 that the PlusRide and Generic/TAK mixes can also be prepared by wet process.

2.1.6 McDonald Mixes

McDonald blend is an A-R blend which is produced by first blending CRM and asphalt (AC 20 or AC 30) in a blending tank, and using the modified binder (obtained by allowing the blend to react for a sufficient period in a holding tank) for mix production. There is also a continuous blending technology that is similar to the McDonald process of blending. However the CRM and asphalt (AC-5 or AC-10) are continuously blended during the mix production or by prepared on hand and stored in storage tanks for later use (1,5).

2.2 HISTORICAL DEVELOPMENT

Asphalt-rubber is produced by combining asphalt and tire rubber with or without the use of distillate additives. Though the component composition of all asphalt rubber blends are essentially equivalent, the product obtained after blending the components is said to vary dramatically (9). This is because, the properties of the blend are influenced by mixing temperature, reaction time, reaction temperature, rubber concentration (10). Hence to get consistent properties stricter controls are required during the preparation of asphalt rubber blends.

The purpose of blending CRM with asphalt was to enhance the elastic and resilient properties of the asphalt. With this objective in mind researchers began by trying different methods to produce A-R blends. Huff and Vallerga (11) have traced the historical stages in the development of the A-R blends. They indicate that the first attempt in this direction was made by adding natural rubber to asphalt. Although good results were obtained, the modified binder indicated an increase in viscosity with an increase in the percentage of natural rubber. Less percentage of rubber was used to reduce the blend viscosity.

The use of natural rubbers resulted in the oxidation of the blend with time. The natural rubber also would be converted into oil on being overheated, thus softening the asphalt. This created a barrier in the large scale production of Asphalt-Rubber. Later on, synthetic rubber, which was less expensive than the natural rubber was used to prepare the A-R blends. However, the synthetic rubber was reported to lack elasticity and tackiness when compared to the natural rubber.

As synthetic rubber became popular, the growing pile of scrap tires was eyed as a cheaper source of rubber to prepare asphalt-rubber blends. These scrap tires could be ground and mixed with hot asphalt in large percentages to produce a material that had properties better than the base asphalt. Huff and Vallerga (11) identified some of the distinct advantages and disadvantages of using synthetic rubber. The advantages are as follows:

1. Scrap rubber, being synthetically compounded and vulcanized to resist heat and overheating, eliminated the problems encountered with virgin polymer.

2. Synthetic rubbers lacked solubility thus, unlike the natural rubber the synthetic rubber does not convert into oil on being overheated. Instead, the synthetic rubber draws oils

out of asphalt to produce swollen gel like rubber particles. These swollen rubber particles knit together within the asphalt matrix to form an A-R sheet which are more resistant to the fracture stresses than the base asphalt itself.

3. Scrap tire rubbers possess valuable components which are often overlooked but might well contribute to the improvement of the asphalt. Some of these are:

Carbon Black : Scrap rubber contains more than 20 percent carbon black, an element that has been shown to add reinforcing properties to asphalt.

Antioxidants: These are said to counteract the weathering of tires and aid in increasing the durability of rubber.

Amines: These are added during the de-vulcanizing processes and are closely related to the anti-stripping compounds. Studies have indicated that the act as anti-stripping agents.

Aromatic oils: these are similar to the rejuvenating agents which prolong the life of asphalt-rubber material.

The disadvantages identified by Huff and Vallerga are:

1. The drawing of oils into rubber particles adversely affects the cohesive and adhesive properties of the asphalt phase. This reduces the binders' ability to bond with pavement surfaces or with the aggregate particles. This problem was solved with the use of very soft asphalt rich in oils. However, the resulting binder would remain soft and tender.

2. Large quantities of rubber (in excess of 20 percent) were required to produce the desired matrix. The resulting blend had a viscosity much too high for most conventional asphalt applications. This problem was solved with the use of kerosene as a cutback. However, the mix became too tender before curing thus limiting the use for chip seal purposes only.

2.3 ASPHALT-RUBBER BINDER PRODUCTION

2.3.1 AZDOT Lab Method of Asphalt-Rubber Production

Pavlovich (12) outlines the methodology adopted by the Arizona DOT for the preparation of A-R blends using unmodified ground crumb rubber produced by mechanically grinding the passenger car treads. The CRM having the gradation given in Table 2-1 was blended with AR-1000 viscosity graded asphalt cement having the properties indicated in Table 2-2 to produce the asphalt-rubber blends. The detailed procedure is as follows:

1. About 750 grams of asphalt is weighed into a 3000 ml stainless steel beaker and the asphalt was heated to the specified mixing temperatures (176, 190 and 204C) with the thermometer placed 6.3 mm from bottom and 12.5 mm from side of beaker.

2. Apart from manual agitation to prevent local overheating, the asphalt is stirred with a three blade propeller at a constant propeller speed of 750 rpm by maintaining a voltage of 115 volts using a powerstat.

Table 2-1 Gradation of CRM Used in AZ DOT Process[12]

Sieve Size (mm)	Percent Passing
1.18	95 - 100
0.5	0 - 10

Table 2-2 Properties of Asphalt Cement Before and After the Research Program[12]

Test Type	May 1978		August 1978	
	AR 1000	After RTFO	AR 1000	After RTFO
Pen (Std)	134	86	138	94
Abs Vis, P 60 C, 30 cm	613	1280	642	1166
Kin Vis., Cst	159	230	155	215
Std Duct. cm	150+	---	134+	134+
Solub, % (TCE)	99.6	99.8	99.7	99.2
Softening Pt, F	104	113	--	--
Sp. Gravity	--	1.0155	--	--

3. When the temperature of asphalt is 14 C below the mixing temperature, the gas flow was lowered to maintain the mixing temperature.
4. When the temperature stabilized, 250 ml of rubber maintained at room temperature was added to the hot asphalt within five seconds. The addition of cold rubber caused the mix temperature to drop by about 28 C below the mixing temperature in about 5 minutes.
5. As the temperature of the blend began to rise, the gas flow and the propeller speed was increased. It is said that the temperature stabilizes at the prescribed mixing temperature in about 30 minutes. At this point, the timing for the prescribed holding time (varies from 0.5, 1.0 and 2.0 hours) is started.
6. During the holding or reaction time, the asphalt-rubber was manually scraped from the sides of the beaker. At the conclusion of the holding period, the burner was removed, the blend was transferred to five 250 ml. marked cans and maintained in a refrigerator at 10C after the blend cooled to the room temperature. The details of the setup used for the preparation of asphalt-rubber is shown in Figure 2-4

2.3.2 FAA Method of Preparing Asphalt-Rubber

The FAA procedure is largely based on the experience from Arizona, New Mexico, and Texas. Roberts (10) reports that the principle underlying the preparation of asphalt-rubber is that the reaction between the asphalt and rubber continues until a stable viscosity is achieved. Even though a stable viscosity can be achieved using a set of mixing times and temperatures, a definite combination is essential in preparing the blends for the mix design.

In the FAA procedure, about 1000 ml. of asphalt is heated using an electronic temperature controlled heater. The asphalt is stirred using a constant speed motor with a propeller stirrer to avoid local overheating. The binder is then transferred to a 2000 ml. reaction flask along with the diluent if included in the mixture. Maintaining the mixer at a speed of 500 rpm, the rubber was added to the asphalt in about 10 seconds as soon as the temperature reached 190 C. The digestion time recorder is started upon addition of rubber. The reaction between asphalt and rubber is continued for not less than an hour or until the output from the stirrer (viscosity) reached an uniform level. After blending, the asphalt rubber is ready for use in mix preparation and storage. The test setup used in the FAA procedure is similar to the AZDOT procedure.

2.3.3 Rouse Rubber Inc. Method of Asphalt-Rubber Production

In the Rouse Rubber Inc. method of preparing Asphalt-Rubber blends (13), UltraFine GF-80 CRM is used. UltraFine GF-80 refers to a CRM gradation with a nominal maximum size of 180µm. The main objective of this procedure is to produce a completely reacted A-R binder by monitoring the viscosity during the reaction period. The reaction time is considered to be a function of temperature. The A-R reaction time is said to decrease by 50 percent for every 11 C increase in temperature. However, the lowest and highest reaction temperatures are 154 and 182 C respectively. The details of A-R production procedure are as follows.

1. The A-R reaction time and temperatures are selected from Table 2-3 and desired amount of asphalt cement is accurately weighed into a stainless steel container. The AC is heated to the pre-blending temperature using a hot oil bath or heat source. The AC is agitated at 20 rpm as it is heated to the pre-blending temperature.
2. The amount of CRM is weighed out as a percentage of the weight of AC and when the asphalt reaches the blending temperature, the CRM is added and dispersed into

Table 2-3 Selection of Reaction Time and Temperatures for A-R Blend Preparation[13]

Application	Percent CRM	Reaction Time (min.)	Temperature (C)
Dense graded	5%	40-50	154
Dense graded	10%	50-60	154
Dense graded	5%	25-35	163
Dense graded	10%	30-40	163
Dense graded	5%	1525	177
Dense graded	10%	20-30	177
Open graded	15%	Not Recommended	154
Open graded	15%	30-40	163
Open graded	15%	20-30	177
ARMI	15%	30-40	163
ARMI	15%	20-30	177
ARMI	25%	Not Recommended	163
ARMI	25%	25-40	177

the asphalt during the next 3-5 minutes. The blending is continued until the end of the reaction time.

3. To evaluate the variation of the viscosity with time during the reaction period, the method recommends the determination of viscosity using a Brookfield viscometer at every minute for the first 10 minutes. After 10 minutes, the viscosity is determined at every 5 minutes for the next 20 minutes, then at 45 minutes, 1, 2, 3, 4, 5 and 24 hours and beyond if needed.

2.3.4 Field Production of Asphalt-Rubber

Asphalt-rubber is produced in the field after incorporating some modifications to the existing asphalt plant. These modifications include a blending accessory, combination of blending and reaction tanks, rubber storage, rubber feed, heated blending tanks (143 to 205 C) and a heated reaction tank (176 to 205 C) (1). The common types of systems used for the production of asphalt-rubber are the Continuous Blending and the Blending/Reaction Systems.

2.3.4.1 Continuous Blending Systems

This system consists of a blending unit with agitators and two 2000 liter retention tanks (1). The CRM in various proportions can be mixed directly with the liquid asphalt in a tank equipped with a large propeller type mixer. Brock (14) indicates that the mixing time ranges from a few minutes to an hour depending upon the particle size of the rubber and the temperature of asphalt. The required reaction time is said to double with every 11 to 14 C reduction in temperature upon the introduction of ambient rubber into the asphalt tank. The temperature reduction has to be counteracted by increasing the temperature of the liquid asphalt using booster heaters prior to the introduction of the cold rubber. The asphalt-rubber storage tank must be equipped with a mixer to enhance circulation in order to prevent coating of hot surfaces. The FHWA Workshop Manual on CRM (5) indicates that the output capacity of these continuous blending systems ranges from 400 to 600 liters depending upon the gradation of the CRM. Figure 2-5 shows the line diagram of Continuous Blending Technology used by Rouse Rubber Industries, Inc.(13)

2.3.4.2 Blending/Reaction Systems

This system consists of a trailer mounted reaction tanker with a modified agitation system and heat system. A heavy duty abrasion-resistant pump is required to handle the high viscosity material and the wear from suspended carbon black particles (1).

2.4 ASPHALT-RUBBER BINDER PROPERTIES

The factors affecting the properties of asphalt-rubber are CRM type, processing method, rubber concentration, gradation of rubber particles, digestion temperature, type and concentration of catalyst, and type and concentration of the extender oil. These factors affect the physical properties like viscosity, ring and ball softening point, elastic recovery of strain and force ductility. For the CRM mixes prepared by the dry process, the CRM properties affects the performance properties of the mixes. A knowledge about the effect of CRM properties on the mixes will help to develop better mix design procedures for the CRM mixes.

2.4.1 Gradation Requirements for A-R Blends

The gradation specifications are different for the rubber modified binders prepared by the dry and wet process. Shuler et. al. (6) reports the use of four gradations of CRM. The details of the gradations are given in Table 2-4. The McDonald A-R blends which are typically constituted by 15 percent of CRM (by weight of asphalt) is so selected that the CRM particles in the blend can be accommodated in the gap produced by the coarse aggregate gradation. The CRM gradation for the Dense-Graded and Open Graded HMA containing A-R binder is given in Table 2-5.

The Arizona DOT specifies (15) gradations for the rubber materials used in the Asphalt-Rubber Stress Absorbing Membrane Interlayer (SAMI) and Asphalt Rubber Stress Absorbing Membrane Seal coat (SAMS) based on the duration of intimate contact between the asphalt and rubber. Table 2-6 indicates that finer rubber gradations are required (to cause the proper physical and chemical bonding) when the duration of intimate contact is less than 5 minutes.

Table 2-4 Rubber Gradation Specifications Reported by Schuler [6]

Sieve Size (mm)	**Percent Passing**			
	Type I	**Type II**	**Type III**	**Type IV**
2.36	100	100		100
1.7	95 - 100	95 - 100		
1.18		70 - 80	100	
0.85			95 - 100	
0.6	0 - 10	5 - 15	60 - 80	60 - 80
0.425	0 - 5	0 - 5		
0.300			0 - 10	15 - 40
0.15				0 - 15

Table 2-5 CRM Gradation for Dense and Open Graded HMA[1]

Sieve Size (mm)	**Percent Passing**	
	Dense Graded	Open Graded
1.7	100	100
1.18	98 - 100	75 - 100
0.6	70 - 100	25 - 60
0.3	10 - 40	0 - 20

0.075	0 - 5	0 - 5

Table 2-6 Aggregate Gradations Recommended by AZDOT from Reaction Time Considerations [15]

Sieve Size (mm)	**Duration < 5 Min.**	**Duration > 5 Min.**
1.18	95	98
0.500	< 10	--

The New York Thruway Authority specifies (16) gradations for CRM used in the Asphalt-Rubber Interlayers (SAMI) depending upon whether Rubber Extender Oil or Kerosene Diluent is used to prepare the Asphalt-Rubber. The details are as given in Table 2-7.

Table 2-7 Recommended CRM Gradations for SAMI And SAMS[1]

Sieve Size (mm)	**Percent Passing**	
	CRM Extender	**Kerosene Diluent**
2.36	100	100
0.6	60 - 80	98 - 100
0.3	15 - 40	0 - 10
0.15	0 - 15	0 - 2

Table 2-7 indicates that the CRM extender can efficiently handle the coarser CRM gradation during A-R reaction when compared to the kerosene diluent. The Texas Department of Highways and Public Transportation has used (17) three different rubber gradations in their demonstration projects on asphalt-rubber. The details are given in Table 2-8

Table 2-8 CRM Gradations Recommended by Texas DOT [17]

Sieve Size (mm)	Percent Passing		
	Rubber A	**Rubber B**	**Rubber C**
2.36	100	100	100
1.7	100	100	99 " 0.5
1.18	65 " 5.6	38 " 2.1	67 " 3.9
0.6	2 " 0.3	8 " 0.6	8 " 1.1
0.425	0.5 " 0.4	4 " 0.4	3 " 0.9
0.300	0	3 " 0.4	1 " 0.6
0.150		0.4 " 0.5	0.2 " 0.4
0.075		0	0
Type of CRM	Whole tire, vulcanized and ambient grind	Tread tire, vulcanized and ambient ground	Whole tire, vulcanized, and cryogenic ground

The Virginia Department of Transportation installed some test sections containing asphalt-rubber concrete using the wet process. The mix design used 17 percent CRM by weight of asphalt cement. The CRM contained 14 percent tire rubber and 3 percent tennis ball rubber. The supplier felt that tennis ball rubber would impart some desirable properties and was hence used in this project (18). The gradation of crumb rubber used in VDOT project is given in Table 2-9.

In the FAA mix design process for the design of Asphalt-Rubber concrete mixes for airports, a fine and a coarse gradation of rubber has been adopted (10) and the details of the gradation are given in the Table 2-10. From the above discussions, it can be seen that the dense graded mixes require finer CRM gradation to accommodate the rubber particles in the mix without affecting the volumetric properties of the mixes.

Table 2-9 CRM Gradation Used in the VDOT Project.[18]

Sieve Size (mm)	Percent Passing
1.7	100
1.18	95 - 100
0.6	70 - 100
0.22	0 - 20
0.075	0 - 5

Table 2-10 Gradation of Scrap Rubber Adopted in FAA Mix Designs[10]

Sieve Size (mm)	Percent Passing	
	Coarser Gradation	**Finer Gradation**
1.7	100	100
1.18	55	85
0.6	5	70
0.3	0	50
0.15	0	8
0.075	0	3

2.4.2 Effect of Rubber Type

From Sections 2.2.2 to 2.2.4 it is evident that the preparation of asphalt-rubber blends involves both physical and chemical reaction between asphalt and rubber. Thus, the chemical properties of both asphalt and rubber are said (12) to affect the properties of the asphalt-rubber and hence those of the asphalt-rubber mixes. Rubber from passenger car tires, truck tires and tennis balls have been used. Depending upon the type of CRM used, the blending method is said to vary.

Brock (14) indicates that the mixes made from automobile tires differ from those made with truck tires. He states that the difference in terms of the viscosity, ring and ball softening point and ductility can in part be related to the chemical balance of rubber in the two tire types. One constituent of tire rubber known to affect the A-R blend behavior is the natural rubber component. Glenn and Tolonen (7) indicate that the whole truck tires contain approximately 18 percent natural rubber compared with 9 percent for whole automobile tires and 2 percent for automobile tire treads.

Huff and Vallerga (11) indicate that asphalt-rubber prepared with vulcanized synthetic rubber (scrap tires) indicated better weather and heat resistant properties when compared to the non-vulcanized rubber. The vulcanized rubber is said to form an asphalt-rubber sheet due to the swelling of rubber after absorbing the oils in asphalt. This is said to impart better resistance to fracture under traffic. The asphalt-rubber prepared with de-vulcanized rubber indicated better dispersion and dissolution in asphalt and better binder properties (adhesion and cohesion). However, these blends are reported to lack the toughness and resilience achieved with the vulcanized asphalt-rubber blends.

2.4.3 Rubber Processing Method

The method adopted to process the scrap rubber significantly affects the digestion of rubber and the properties of asphalt-rubber and its mixes. Oliver (19) reports that an electron micrograph study on the rubber particles indicated that the rubber processing method affects the rubber size and shape of rubber particles. The processing method, therefore, affects the surface area of the rubber particles, which in turn affects the rate of reaction and viscosity (7) of the asphalt-rubber binder

2.4.4 Rubber Concentration and Particle Size of Rubber

The size of rubber particles affects the characteristics of mixes prepared by dry and wet process. The size of rubber particles affect the extent of asphalt-rubber reaction in the wet process, with the coarser rubber particles reacting less than the finer particles. The gradation of rubber particles are specified for the preparation of asphalt-rubber. In addition, the gradation of CRM must be so chosen that any unreacted CRM will fit into the space provided by the VMA. Unreacted CRM can render the mix spongy and will affect the air-voids. Hence the mix performance in the field can be significantly influenced by particle size and gradation(2).

Khedayi et al. (20) evaluated three gradations of rubber at four different concentrations, and using five asphalt contents. Their objective was to identify the effect of rubber concentration and gradation on conventional physical properties of the binders. They concluded that the addition of CRM to asphalt inversely affects the penetration, ductility, and flash point, while directly affecting the softening point. In addition, they have reported a decrease in ductility and specific gravity as the gradation of CRM got coarser.

2.5 PREPARATION OF PLUSRIDE CRM MIXES

Until recently, the design of CRM mixes was being mainly accomplished by the conventional Marshall Method with or without relaxation in the specifications depending on the problems posed upon incorporation of CRM (2). But with the development of Superpave technology, researchers have attempted to design the rubber modified mixes using the volumetric method even though the Superpave mix design methods were not developed to do the same. Most of the research conducted on CRM mixes is based on mixes designed using the Marshall method. This section will review the mix designs processes followed by various researchers, State DOT"s and CRM mix producers.

2.5.1 Design Considerations for PlusRide CRM Mixes

PlusRide mix is the trade name of the mix marketed under patent by the Swedish companies Skega AB and ABVaegfoerbaettringar (ABV). Being a patented mix, three types of aggregate gradations are supplied by the patent company for the design of Gap Graded RUMAC Mixes. These are named as PlusRide 8, PlusRide 12 and PlusRide 16 gradations.

2.5.1.1 Aggregate Gradations Used in PlusRide Mixes

Esch (21) reports that the aggregates are gap graded in the range 3.1 to 6.3 mm size to accommodate the fine and ground rubber. Figure 2-2 shows the gap gradation for a typical PlusRide II mix. Absence of gap gradation will result in the rubber particles resisting mix compaction during rolling and the result is an asphalt layer exhibiting excessive air voids and low durability. Based on experience, three different aggregate gradations have been recommended to serve different traffic levels. The details of aggregate gradation used in Gap Graded Plus Ride RUMAC mixes are given in Table 2-11. The Alaska DOT & PF was among the first States to use the PlusRide Mixes in the United States. Five experimental projects were constructed between 1979 and 1983 using the PlusRide Technology (2). Slight variation in aggregate gradations were permitted to provide flexibility to the contractor in the selection of final gradation (21,22). The details of the aggregate and rubber gradations used in the projects are given in Tables 2-12 and 2-13.

Table 2-11 Aggregate Gradation for Gap Graded PlusRide RUMAC Mixes[1,9]

Passing Sieve Size (mm)	PlusRide 8	PlusRide 12	PlusRide16
19	-	-	100
15.8	-	100	-
12.5	100	60-80	50-62
9.5	60-80	30-44	30-44
1.70	23-38	20-32	20-32
0.600	15-27	13-25	12-23
0.075	08-12	08-12	07-11
% AC Mix Wt.	8.0-9.5	7.5-9.0	7.5-9.0

Table 2-12 Aggregate Gradations Used in Alaska DOT & PF PlusRide Mixes[21,22]

Sieve Size (mm)	Carnation 1979	Seward 1980	Peger 1981	Huffman 1981	Lemon 1983
19	100	100	100	-	-
15.8	-	-	-	-	100
12.5	-	78-94	-	-	-
9.5	60-77	43-57	53-67	100	62-76
6.3	-	-	-	-	32-42
4.75	45-59	29-43	28-42	47-60	-
1.70	29-41	22-34	20-32	30-42	22-32

0.600	12-20	15-23	14-22	15-24	20-25
0.075	4-10	5-11	5-11	5-11	5-11

Table 2-13 CRM Gradations Used in Alaska DOT & PF PlusRide Mixes[21,22]

Passing Sieve Size (mm)	Alaska 1979-80	Alaska 1981	Alaska 1983	ABV Coarse & Fine	PlusRide 1981
6.3	---	---	100	100	---
4.75	100	100	76-100	76-92	100
1.70	15-35	15-36	28-36	28-36	28-40
0.850	---	10-25	10-24	10-24	---
0.425	0-6	---	---	---	0-6
0.075	0-2	---	---	---	---

The Minnesota Department of Transportation tried (23) the gap graded "PlusRide" mixes in wearing courses in their demonstration projects for ice and snow control purposes as an alternative to the use of chemicals. The aggregate and the rubber gradations used in these project are given in Table 2-14

Table 2-14 Comparison of MnDOT and Patented PlusRide Aggregate Gradations[23]

% Passing Sieve Size (mm)	MnDOT	PlusRide 8
15.8	100	100
9.5	60 - 80	100
4.75	30 - 40	60 - 80
1.70	20 - 32	23 - 38
0.600	13 - 25	15 - 27
0.475	08 - 12	08 - 12

The gap gradation is enforced such that not more than 10% of the total sample passing

4.75 mm sieve is retained on 2 mm sieve. In other words, passing 4.75 mm sieve and retained 2 mm sieve is 10% maximum. Mineral filler is required to meet the high 75 μm requirements, and that the type and quantity of mineral filler used in the production must be used in the mix design. Since PlusRide II mixes exhibit better resilient/elastic properties compared to the conventional asphalt mixes, the conventional stability and flow criteria does not apply to the mix design. The granulated rubber ground from the passenger or truck tires with a maximum length of 8 mm has been used at a rate of 3% by total weight of the mixture (22). The gradation of the rubber is given in Table 2-15

Table 2-15 Comparison of MnDOT and Patented PlusRide CRM Gradations [23]

Passing Sieve Size (mm)	MnDOT Gradation	PlusRide
6.3	100	100
4.75	76 - 100	76-88
1.70	28 - 42	28-42
0.850	16 - 24	16-42

2.5.1.2 CRM Gradations Used in PlusRide Mixes

The CRM used in the PlusRide mixes can range from 1% to 6% by weight of the total mix, with 3% rubber being commonly used (8). The gradation of rubber used in PlusRide mix has undergone changes since its first use in the late 70s. Initially, only the coarse rubber grading was being used by the patent company. Experience with the mix indicated better durability with an increase in the fine rubber content. Hence, after 1981, 20% of the originally used coarse rubber grading was replaced with finely ground crumb rubber (passing-850 μm sieve) (21,22). Table 2-16 shows the most recent CRM gradation used by the patent company.

2.5.1.3 Range of Optimum Asphalt Contents Used in PlusRide Mixes

Normal paving grade asphalt is used for the PlusRide Mixes. However, the required asphalt content is 1.5 to 3% higher than the conventional dense-graded mixtures. For mix designs the trial asphalt contents are selected by rule of thumb, as approximately 2% more asphalt than a conventional mixture with similar size and type of aggregates (25). The range of asphalt content used in PlusRide mixes are given in Table 2-17

Table 2-16 Current and Original Rubber Gradations Used in PlusRide Mixes[24]

% Passing Sieve Size (mm)	Current	Original (1981)
6.3	100	---
4.75	76-88	100
0.425	28-42	28-40
0.850	16-42	---
0.425	---	0-6

Table 2-17 Range of Asphalt Contents Used in PlusRide Mixes[25]

PlusRide Mix Designation	Range of Optimum AC Used
PlusRide 8	8.0-9.5
PlusRide 12	7.5-9.0
PlusRide 16	7.5-9.0

2.5.1.4 Preparation of PlusRide Mixes

PlusRide mix is a patented mix thus requiring the paying of royalties. PlusRide mix samples are prepared using Marshall molds with suitable modifications to the material and mold handling procedures. The following procedure has been identified by researchers (1,6,8,22,23,24,25), to prepare the PlusRide mix samples.

1. The aggregate fractions for the selected gradation are combined in pre-calculated

quantities and placed in an oven at a temperature of 190 to 218 C for at least 12 hours and the asphalt used for the mix preparation is maintained at 135 C prior to the mixing.

2. The rubber fractions are combined to produce the desired gradation and weight. Normally 3% (1 - 6%) of rubber is used in case of PlusRide mixes and 2% for surface and 4% for binder courses in case of Generic mixes (24) The rubber percentage is expressed in terms of the total weight of the aggregates.

3. The heated aggregates are mixed with the rubber granules and placed in an oven at 190 C or 218 C for approximately 15 seconds (1,6). It must be noted that the temperature of 218 C has been adopted to increase the potential for dissolving some of the fine rubber into the asphalt. This is said to improve the resilient modulus and fatigue life (25).

4. The required amount of asphalt maintained at a minimum temperature of 135 C (Max. 160C) is added to the aggregate rubber blend and mixed for 2-3 minutes (1, 23) to yield a mix having an uniform distribution of asphalt throughout. A curing period of 1 hour at 160C is adopted for the PlusRide mixes and no such curing period is recommended for TAK mixes (8)

5. The heated mix is then compacted in standard Marshall molds (100 mm diameter and 62.5 mm height) maintained at 135 C. These molds are to be coated with silicone grease to cause easy removal of the specimen from the mold (1,8)

6. The mix is compacted at a temperature of 129.5 C with 50 blows for PlusRide Mixes (8,23) and 50 or 75 Blows (8,24) for TAK mixes from Marshall hammer on either sides.

7. The base plate is to be removed immediately after the compaction and the mold containing the mix is placed on a wooden plug of 98 mm diameter by 25 mm thick wooden plug. Another wooden plug is placed on the top of the specimen, weighted

(2.2 Kg.) and allowed to cool or maintained for 24 hours before extrusion (8,23,25).

8. The specimens are removed from the mold at room temperature by means of an extrusion jack and then placed on a smooth, level surface until ready for testing.

9. The bulk specific gravity and height of the specimens are measured immediately after extruding from the mold.

2.5.1.5 Mix Design Criteria for PlusRide Mixes

Kandhal and Hanson (24) indicate that the design criteria for PlusRide mixes is to determine an aggregate gradation, AC and CRM content that yields a mix having :

1. High-coarse aggregate content, gap graded to provide space for rubber granules to form a dense, durable and stable mixture upon compaction.
2. A rich asphalt/filler ratio to ensure a workable mixture and durable pavement.
3. A low void content in the compacted mix. The voids should be in the range of 2 to 4 percent, with 3% being normal.

Chehovits et. al. (8) have indicated some additional mix design criteria for the PlusRide Type Mixes in Table 2-18

Table 2-18 Mix Design Criteria for PlusRide Mixes[8]

Property	Value
Voids (%)	2 - 4%
Min. Modulus psi @ 25 C ASTM D 4123	100,000
Retained Strength (%) AASHTO T 283	75

2.6 DESIGN OF CRM MIXES BY TAK/GENERIC METHOD

The TAK System/Generic Dry Technology uses the conventional dense gradation. CRM is added to the conventional dense aggregate gradation to produce a dense graded Rubber Modified Asphalt Concrete (RUMAC) mix (1). The gradation of CRM affects the asphalt-rubber reaction and hence the characteristics of TAK mixes. In the dry process of preparing the TAK mixes, the gradation of rubber is so selected that the coarse rubber particles will serve as elastic aggregates and the fine rubber will react with asphalt to produce modified binder. The gradation requirements of CRM are however different for the PlusRide and TAK mixes (8).

The TAK/Generic RUMAC is a two component system, CRM passing 850 micron sieve is believed to react with the asphalt cement to produce a modified binder and the coarse CRM serves to replace a portion of the aggregates in the HMA mixture and act as an elastic aggregate. The aggregate gradation is the key to successful to RUMAC projects. If CRM gradation is coarse or the aggregate gradation is too fine, the mix would pose compaction problems. In all the cases the CRM should be considered as a part of the void space. If the void space is inadequate for the CRM, early pavement performance problems will be experienced (8). Inadequate void space for the rubber particles could result in large variations in the void content at same asphalt content, constant air voids with increasing asphalt content, and expansion/swelling of the specimen after compaction.

Chehovits et. al (8) indicate that the above problems have been addressed by reducing the size of crumb rubber or by opening up the aggregate gradation. The aggregate gradation must be selected by first identifying whether or not the CRM can be incorporated into the void provided by the aggregate gradation. Consideration must be given to the fact that the CRM swells after it comes in contact with the asphalt cement during mixing, hauling, placement and compaction. The size of the CRM is kept one sieve size smaller than the gap existing in the mineral aggregate.

2.6.1 Aggregate Gradations Used in Generic/TAK Mixes

A conventional dense-graded aggregate gradation is used with slight modification to accommodate the rubber particles. There is very limited information about the gradation as to how the amount and gradation of CRM is determined for a specific mineral aggregate. The aggregate gradation used in dense graded RUMAC must be on the coarser side of the specification to accommodate the CRM (8,24). The recommended aggregate gradations for TAK/Generic Mixes are given in Table 2-19

2.6.2 Gradation of CRM Used in TAK Mixes

The CRM used in TAK/Generic system is a two component system in which the fine crumb rubber interacts with the asphalt cement and the coarse crumb rubber functions as an elastic aggregate in the HMA mixture. Generally, one to three percent crumb rubber (weight of HMA mix) and asphalt content of 7.5% has been used in the preparation of the TAK or the Generic Dry Mixes (8). The recommended gradation for CRM is given in Table 2-20

Table 2-19 Recommended Aggregate Gradations for TAK/Generic Mixes[8]

Sieve Size (mm)	Nominal Maximum Size (mm)

	19.5 mm	12.5 mm	9.5 mm
25	100	-	-
19	90-100	100	-
12.5	-	90-100	100
9.5	56-80	-	90-100
4.75	35-65	44-74	55-85
2.36	23-49	28-58	32-67
1.18	-	-	-
0.6	-	-	-
0.3	5-19	5-21	7-23
0.15	-	-	-
0.075	2-8	2-10	2-10

Table 2-20 Recommended CRM Gradation for TAK/Generic Mixes[8]

Sieve Size (mm)	Percent Passing
4.75	100
2.36	70-100
1.18	40-65
0.6	20-35
0.3	5-15

The New York Department of Transportation (8) constructed experimental sections using Generic Dry (TAK) Technology with 1, 2 and 3 percent CRM by weight of the total mix. The combined aggregate of the aggregate CRM blend and that of the CRM used in the New York Project are given in Tables 2-21 and 2-22

Table 2-21 Combined Aggregate and CRM Gradations Used in NYDOT Projects on TAK/Generic Mixes[8]

Sieve Size (mm)	Percent Passing	Tolerance (Percent)
25	100	-
12.5	95-100	-
6.3	65-85	+7
3.1	36-65	+7
0.85	15-39	+7
0.425	8-27	+7
0.220	4-16	+4
0.075	2-6	+2

Table 2-22 CRM Gradations Used in NYDOT Projects with TAK/Generic Mixes[8]

Sieve Size (mm)	Percent Passing	
	Specified	**Supplied**
6.3	100	-
4.75	-	100

3.1	75-85	-
1.7	45-55	51
0.850	30-40	44
0.425	0-10	19

2.6.3 Preparation of TAK/Generic Mixes

The sample preparation or sample fabrication steps are almost the same for both PlusRide and Takkalou Mixes but for the gradation of aggregates and CRM and the mix curing period after mixing the aggregate and CRM with asphalt. The Takkalou System (20) of production of rubber modified asphalt mixes uses a standard dense-graded aggregate whereas the patented PlusRide mix uses a unique or gap graded mix. The Takkalou mix is produced by adding the coarse and fine rubber to the hot aggregates and mixing at prescribed temperature. The hot asphalt is then added to this aggregate-rubber blend and mixed intimately. The intimate mixing is believed to cause an increase in the viscosity of the binder when the fine crumb rubber particles reach optimum swelling. Thus, the role of rubber is to increase the viscosity of the binder (fine rubber) and to act as an elastic aggregate (coarse rubber) to improve the elastic properties of the mix and reduce the temperature susceptibility (21).

2.6.4 Mix Design Criteria for TAK/Generic Mixes

The mix design of Generic RUMAC involves the establishment of CRM content which meets the agency's minimum stability requirement. Generally, up to 2% CRM is used in surface courses and up to 4% in base courses. The combined gradation of aggregate and CRM is determined by using a weight adjustment factor of 2.3 for CRM to account for the differences between the specific gravity of aggregates and rubber. After selecting the amount and gradation of CRM, trial specimens are made with 50 or 75 blows of Marshall hammer or by kneading compaction. Table 2-23 gives the criteria for determining the Optimum Asphalt Content for TAK mixes.

Table 2-23 Comparison of Design Criteria for Gap-Graded PlusRide and Dense-Graded TAK Mixes[23]

Criteria	PlusRide	TAK
Compaction	75 Blows/Side	50 Blows/Side
Air Voids (%)	2-4%	3 - 5%
Minimum Stability	1800 lb. (min.)	800 lb.
Flow (0.1")	< 20	8 - 20
VMA (% min.)	17	--
Retained Strength (%) - AASHTO T283	> 75	--

2.7 PERFORMANCE EVALUATION OF CRM MIXES

Takkalou Et. al (25) have used 3% CRM content to evaluate the effect of rubber gradation, air voids, aggregate gradation, mix temperature and curing conditions on the properties of TAK and PlusRide mixes. In all, 26 combination of mixes to evaluate the effects of rubber gradation, content, air voids, aggregate gradation, mix temperature, and curing conditions on the properties of rubber modified mixes. Coarse rubber, fine rubber, and three blends of coarse and fine rubber were used in their research program. The details of the aggregate and CRM gradation used in the laboratory research program are given in Tables 2-24 and 2-25.

Table 2-24 Aggregate Gradations Used by Takkalou Research[25]

Sieve Size (mm)	Gap-graded	Dense- graded	PlusRide 12
19		100	-
15.6	100	-	-
9.5	70	76	60-80
6.3	37	-	30-42
4.75	-	55	-
1.7	26	36	19-32
0.6	18	-	13-25
0.425	-	22	-
0.075	10	7	8-12

Table 2-25 CRM Gradations Used by Takkalou Et. al [25]

Sieve Size (mm)	CRM Gradations				
	Coarse	**Fine**	**80/20**	**60/40**	**80/20**
6.3	100	100	100	100	100
4.75	97	100	97.6	98.2	76 - 92
1.7	15	100	32	49	28 - 36
0.85	4	86	20.4	36.8	10 - 24
0.425	3	30	8.4	13.8	--------
0.300	2.9	20	6.3	9.7	--------

For mix designs the trial asphalt contents are selected by rule of thumb, being approximately 2% more asphalt than the conventional mixture of similar size and type aggregates (24). The sensitivity of the PlusRide mixes to asphalt content was studied (22) by performing mix designs using a single aggregate source, an AC 2.5 asphalt and a rubber content of 3%. Test specimens were prepared using four Aggregate gradations corresponding to - Coarse (A), Fine (B), Mid Point (C) and Straightest Line (D) within the specification band. For each gradation, the specimen asphalt contents were 6, 7 and 8 percentage (by dry weight of the aggregates) and the CRM content was 3 percent. The aggregate gradations bands were slightly wider than those recommended for the similar "PlusRide 12" mix. The aggregate gradation are shown in Figure 2-6

2.7.1 Effect of Aggregate Gradation and AC Content on Mix Properties

Based on the above study by Esch (22) the following conclusions were drawn about the effect of aggregate and AC content on the mix properties.

1. For all the four gradations, the percentage voids decreased with an increase in asphalt

content.

2. The fine and coarse gradation indicated minimum and maximum voids respectively (Figure 2-7).

3. The finer gradation indicated maximum stability (compared to other gradations) at all asphalt contents (Figure 2-8)

4. Fine gradation indicated maximum flow compared to other gradations at asphalt contents of 8 and 9% (Figure 2-9)

Takkalou et. al (25) have used asphalt contents ranging from 7 to 9.3% depending upon the rubber blend, mixing and compaction temperature, curing period, and surcharge load applied before the sample extrusion. The test results will be discussed in the subsequent articles.

2.7.2 Sensitivity of the PlusRide Mixes to CRM Content

The sensitivity of PlusRide mixes to rubber content was evaluated (22) using Marshal specimens prepared using four aggregate gradations, three AC contents (6,7 and 8%) and three CRM contents (2.5, 3.0 and 3.5 percent). The studies indicated that:

1. A 1/2 percent variation in rubber content may cause a change in Marshal stability by 10 to 30% of the original stability. (Figure 2-8)
2. A 1/2 percent change in rubber content can cause a 1% change in air voids at the same asphalt content and would require a 1% change in the design asphalt content to reach the same air-void level. (Figure 2-7)
3. The PlusRide mixes are very sensitive to rubber content and it appears that a 2.5% target rubber content may be much more economical than the normally recommended 3% rubber content (Figure 2-8)
4. Close control of the rubber addition is essential to obtain consistent mix behavior since stability and voids vary considerably with small changes in rubber content.This suggests that the mix production should be restricted to batch plants where rubber content can be accurately controlled.

Takkalou Et. al (25) have studied the effect of rubber content and their gradation on the resilient modulus and fatigue characteristics of PlusRide-12 using the mid band gradation. Marshal specimens prepared with rubber percentages of 2 and 3%, corresponding to coarse, fine and medium (60/40 ratio) gradation were tested for resilient modulus and fatigue at 10C, to determine the effect of aggregate gradation, air voids, aggregate gradation, mix temperature and curing conditions on the properties of TAK and PlusRide mixes. The results indicated the following:

1. The fine gradation indicated the highest resilient modulus and least fatigue life compared to coarse and medium gradations. (Figure 2-10 and Figure 2-11)

2. The resilient modulus and fatigue life of medium rubber and fine gradation are comparable (Figures 2-10 and 2-11).

3. Mixes with 2% rubber content indicated higher resilient modulus at 10 C (for all fine, coarse and medium gradations) compared to the mixes with 3% rubber content (Figure 2-12)

4. No appreciable increase in fatigue life is indicated by increasing the CRM content to 3 percent for coarse and medium gradation of rubber (Figure 2-13). However, the fine rubber gradation indicated a substantial increase in fatigue life with an increase in rubber content from 2 to 3 percent (by weight of the aggregates).

2.7.3 Effect of Fine Rubber and Curing Practices

The fine rubber in the PlusRide mixes reacts with the asphalt cement to produce a modified binder which imparts superior structural properties to the mix in terms of fatigue and resilient modulus. Laboratory studies were performed at the Anchorage Central Materials and Fairbanks Research Laboratories (22) to evaluate the effect of fine CRM content and curing period on the fatigue and resilient modulus characteristics of the mix.

Marshall specimens were prepared using two aggregate gradations using AC 2.5 Asphalt. One half of the samples were mixed at 190.5 C and compacted at 121 C. To the other half was added an additional 2 percent fine rubber (850μ sieve). These samples were heated to 204 C and cured in an oven for 45 minutes in closed containers following compaction. The specimens were tested for resilient modulus and fatigue properties using the diametral loading device at 1 loading cycle per second with a load duration of 0.1 second. The results indicated that the samples cured at 204 C and with an extra 2 percent fine rubber content showed an increase in resilient modulus and fatigue life by up to 40 percent and 450 percent respectively when compared to the samples prepared using the existing specifications. (Figure 2-14 and

Figure 2-15)

2.7.4 Effect of Curing Period and Surcharge

Studies by Takkalou et. al (25) indicated that the dense graded TAK/Generic Mixes indicated an increase in resilient modulus with a cure period of 2 hours. However, the effect of curing period was not significant for the PlusRide mixes. The fatigue life of the mixes decreased with a cure period of 2 hours. (Figure 2-16 and Figure 2-17). Also, the dense graded TAK mixes showed an increase in resilient modulus and significant reduction in fatigue life with surcharge loads. (Figures 2-16 and 2-17)

2.7.5 Effect of Mixing Temperature

Studies (25) to determine the effect of mixing temperature on the structural properties of PlusRide and TAK mixes indicated that:

1. High mixing temperature slightly increases the resilient modulus and fatigue life of gap graded mixes tested at 5.5 C. Dense graded TAK mixes showed an increase in modulus, but a decrease in fatigue life with higher mixing temperature. (Figure 2-18 and 2-19)

The effect of cure time after mixing, on both resilient modulus and fatigue life at both curing temperatures (190.5 C and 218 C) for gap gradations was not significant. (Figures 2-18 and 2-19)

2.7.6 PlusRide vs. TAK/Generic RUMAC Mixes

The Generic RUMAC is a two component system, the CRM passing 850μm reacts with the asphalt cement to produce a modified binder and coarse CRM replaces a portion of the aggregates in the HMA mixture, and acts as an elastic aggregate. The TAK or Generic mixes use equivalent or slightly lower percentage of CRM compared to the PlusRide. The CRM is also finer than that used in the PlusRide. Although Chehovits et al (8) indicate through Figures 2-20 and 2-21, that the TAK mixes offer higher fatigue and rutting resistance when compared to the conventional mixes, the PlusRide or TAK mixes may not always provide the best structural properties in all aspects compared to the conventional mixes. Studies (25) indicate that the conventional mixes with no rubber have shown higher modulus compared to the dense graded TAK mixes and mid point gradation PlusRide-12 (both with 3 percent rubber and 80/20 blend). However, the fatigue properties of the PlusRide and TAK mixes are higher compared to the conventional mix. Figure 2-22 and 2-23 (21,25) illustrate this finding.

The PlusRide mixes are reported (22) to offer higher fatigue life due to the modified asphalt binder and elastomeric aggregate. Studies conducted at the Oregon State University (6) indicate that the fatigue strength of PlusRide Mixes is maximum when compared to the conventional gravel and basaltic aggregate gradations (Figure 2-24). The rubber particles in the PlusRide mixes are said to absorb the stresses at the tip of the crack, thereby increasing the resistance to reflective cracking. In addition, laboratory studies have indicated increased resistance to low temperature cracking (6). The rubber granules exposed to the surface is said to compress slightly when subjected to traffic and wheel loads. This creates a small area of flexibility which makes the crystallization of ice difficult. However, the pavement must be loaded continuously and the ice must be relatively thin (24). The MnDOT which uses substantial amount of chemicals for ice and snow control tried the PlusRide mix as an alternate method to control the ice accumulation on the roadway surface. However, no significant de-icing benefits have been reported with the use of PlusRide mixes (23). Increased rutting resistance is possible due to greater resilience offered by the rubber particles. One laboratory research attributes the increased rutting resistance to the rubber and the associated 1.5% increase in asphalt content (24).

2.8 FIELD PRODUCTION OF CRM MIXES

Literature (21,26) indicates that the batch mixing plants are preferred to continuous-mix and drum-dryer mix asphalt paving plants. This is because, required quantities of rubber, asphalt and aggregates can be measured exactly and added to the pug mill or mixing chamber. The use of pre-weighed sacks of rubber in batch-mixing eliminates the need for having a separate bin and a belt feed (as in case of continuous-mix plants) thus offers a better control on the quality of mix production. Esch, Takkalou et. al. and Harvey et. al. (23,26,29) have indicated that strict control need to be maintained on the mixing temperatures. The recommended range of temperatures by Harvey and Curtis (23) are 163C (Max.) for bituminous materials, 163 to 190.5 C for aggregates and a discharge temperature of 163 to 182 C and 135 to 163 C for batch and drum mix plants respectively.

To prevent rapid cooling, the paving mix has to be covered with canvas and the mix is required to be placed on a dry pavement surface at a temperature not less than 149 C in case of batch plant produced mix and 135 C in case of mix produced in drum mixer. In any case, the ambient temperature of the mix must never be less than 7.2 C (26).

Rolling of the mix must start as early as possible after the mix placement and must continue until the mix temperature cools below 60 C. The rubber mixes being very resilient, require the use of steel-wheel static or vibratory type of rolling (21) and the use of detergent based liquids (1,5) in the haul trucks and on the steel rollers during mix compaction. However, experiences with rubber-asphalt pavements placed in the Vancouver, B.C., and Anchorage, Alaska, in 1981 have indicated (21) that significant surface tightness could be achieved with the use of a rubber-tire roller after the mix has cooled below 60 C.

2.8.1 Problems Associated During Mixing

Even though batch, continuous and drum- dryer plants mix asphalt plants have been used without difficulty, Researchers (21,26) have indicated that the use of continuous-mix and drum dryer plants requires the continuous addition of rubber from a separate bin with belt feed to maintain the uniformity and that close control of rubber content is critical to assure proper field performance. It has also been reported (22) that the control of rubber feeding is less accurate with the continuous and drum dryer plants.

Also, potential for producing the smoke has been reported on a single-entry drum mixer due to the removal of the flame heat shield from the drum. It was suggested that this problem can be eliminated with the use of double (mid entry) type that allow the rubber to be added in the center of the mixing drum.

Lowering of the mixing temperature from 162 to 152 C have resulted in the asphalt mix sticking to the flights, which caused the trunion to slip with the increased load. The slippage was also due to the some of the rubber granules blowing from the feeder belt into the trunion. This problem was however been corrected by cleaning the trunion and elevating the temperature back to 165C .

Literature (23,27) indicates that the batch mixing plants are preferred to continuous-mix and drum-dryer mix asphalt paving plants. This is because required quantities of rubber, asphalt and aggregates can be measured exactly and added to the pugmill or mixing chamber. The use of pre-weighed sacks of rubber in batch-mixing eliminates the need for having a separate bin and a belt feed as in the case of continuous-mix plants thereby it offers a better control on the quality of mix production.

2.8.2 Hauling, Placing and Compaction Problems

One of the major concerns with the hauling, placing and compaction of rubber mixes is the temperature. The temperature of the mix not only affects the mix workability but also influences the reaction between the asphalt and rubber. This will result in a modified binder with higher viscosity and impart superior structural properties to the mix.. The following steps have been recommended to assure that proper temperatures is maintained:

1. The hot paving mix transported on trucks must be covered with canvas to prevent rapid cooling.
2. The mix is required to be placed on a dry pavement surface at a temperature not less than 149 C in case of batch plant produced and 135 C in case of mix produced in the drum mixer.
3. The rolling of mix must start as early as possible after the mix placement and must continue until the mix temperature cools below 60 C. This is to counteract the swelling of the mix.
4. The rubber mixes being very resilient, steel-wheel static or vibratory type of rolling is recommended (21). Pneumatic rollers are not usually recommended due to the sticking of the mix on to the wheels.

In addition, only detergent based release agents must be used on haul trucks and rollers (1,24). However, it may be noted that the above problem has been noted with the PlusRide mixes and that the pneumatic rollers have been used with out any problems in the construction of TAK Mixes for the New York Projects (24).

2.8.3 <u>Problems Faced with the Lab Preparation of RUMAC Mixes</u>

One of the problems reported (8,23,25) with the preparation of the PlusRide RUMAC Mixes is the swelling of the compacted specimen if removed immediately after the cooling. The swelling of the compacted specimens is due to the reaction between the asphalt and the fine rubber particles. This swelling of the mix could affect the air voids and the stability of the mix. The problem has been solved by:

1. Removing the base plate immediately after the mix compaction and setting the mold over a 98 mm diameter by 25 mm thick wooden plug. Another wooden plug is placed

on the top of the specimen, weighted (2.2 Kgs) and allowed to cool (24).

2. Similar procedure has been followed by researchers (25) in the preparation of TAK mixes, wherein the compacted molds were subjected to a surcharge load of 2.2 Kgs immediately after compaction. The surcharge was maintained for 24 hours and the samples were then extruded. The other problem faced with the specimen preparation is the sticking of mixes to the mold and filter paper. This problem has been solved by using release paper or greased filter paper or by greasing the base plates, compaction molds and the compaction hammer before the sample preparation.

CHAPTER 3

EXAMINATION OF THE EFFECT OF CRM ON BINDER PROPERTIES

The Asphalt-Rubber binders are modified binders obtained by blending CRM with the conventional binders. These modified binders are constituted with CRM particles which renders them more viscous than the original binders. The application of conventional viscosity or ductility tests to evaluate their consistency does not seem to work because of the heterogeneous property of the binder. Bob Gossett (28) indicates that the capillary tube used to measure the viscosity can become clogged due to the viscous nature of the binder and that the reported results are not consistent.

Heitzman (1) reports that the incorporation of CRM into asphalt and asphalt mixes enhances the rutting and thermal cracking resistance of the mixes but the conventional tests to evaluate the rheological properties of the asphalt-rubber binders do not relate to rutting or fatigue or thermal cracking resistance. Even if it were possible to evaluate the rheological properties of the binders using the conventional tests, these rheological test parameters do not have a practical significance. This is because they do not provide any information about the performance related properties of the binder (29). This calls for the need to identify the rheological properties of A-R blends that can be related to the performance properties.

The asphalt research program under the Strategic Highway Research Program (SHRP) addressed the issue of measuring the rheological properties of the binders and relating those properties to the performance of the binder in the field. Key instrumentation was developed for this purpose to evaluate properties like pumpability, rutting resistance, fatigue, and thermal cracking. Although the Superpave binder specifications were developed for unmodified/virgin asphalt, these specifications will still be used in this study for evaluating the asphalt-rubber binders (30). This section discusses the Superpave rheological properties of the binders, the instrumentation used to measure these properties and the use of Superpave Binder Specifications for Performance Grade (PG) classification.

3.1 PHILOSOPHY BEHIND SUPERPAVE BINDER SPECIFICATION

The Superpave binder specification represents a clear departure from the conventional methods of evaluating the binders. These specifications are based on fundamental measurements obtained at upper, middle and lower range of service temperatures, and are related to rutting, load associated fatigue cracking and thermal cracking. They also consider the aging or hardening of the binders that occurs during mixing, lay down, and service. The use of Superpave binder specification allows the selection or classification of binder from critical (low and high) temperature conditions in comparison to the empirical nature of the conventional viscosity-penetration grading (29) method.

3.2 USE OF RHEOLOGICAL PROPERTIES FOR PERFORMANCE GRADE (PG) CLASSIFICATION

Classifying the binders for Performance Grade (PG) is the main objective of the Superpave Binder Specification. While the conventional viscosity-penetration method of grading the binder is based on the viscosity or penetration values, the PG classification identifies the suitability of the binder for the anticipated maximum and minimum pavement temperatures. In another words, the PG specification answers the question "***do the asphalt properties meet the specification criteria at the critical pavement temperatures?***" (29). In the Superpave binder specifications three temperatures high, intermediate and low are considered. The ***high pavement temperature*** is the average 7-day maximum pavement design temperature and the properties of the binder at high temperature is related to the contribution of the binder to rutting. The ***low temperature*** is the minimum pavement design temperature and the properties of binder at ***low temperature*** is related to the contribution of the binder to thermal cracking. The

intermediate temperature is related to the in-service temperature of the pavement between the two temperature extremes, and the properties of the binder at the ***intermediate temperature*** is related to the load-associated fatigue resistance of the binder.

The properties of the binder used in the Superpave binder specification is the same for all binders, the test temperatures at which these properties are met differ depending upon the grade of the binder. For example: Table 3-1 reproduced from Cominsky et al. (30) shows that irrespective of the binder grade used, the creep stiffness of the binder must not exceed 300 MPa, but the temperatures at which the binder must meet this criteria can vary from 0 to -36 C.

3.3 INSTRUMENTATION TO MEASURE THE SUPERPAVE BINDER PROPERTIES

Brookfield viscometer is used to evaluate the pumpability of the binder. Testing is conducted at 135 C and 20 rpm. The basic principle of a Brookfield Viscometer is that a spindle of known dimension is made to shear a sample of 10.2 gram of binder placed in a cylindrical steel tube. The shear resistance and the spindle characteristics are used to evaluate the Brookfield Viscosity of the binder. Figure 3-1 shows the basic principle of operation of Brookfield viscometer.

The parameters related to rutting ($G^*/\sin\delta$) and load associated fatigue cracking ($G^* \sin\delta$) are measured at high and intermediate test temperatures (at 10 rad/sec) using the Dynamic Shear Rheometer (DSR). The DSR consists of two circular plates between which a sample of binder is sandwiched with a specified gap. The binder is subjected to shear stress at a speed of rotation of 10 rad/sec at the test temperature. The applied shear stress (τ_{max}) and the resulting shear strain (γ_{max}) are measured to determine the Complex Shear Modulus (G^*). The DSR also measures the Phase Angle (δ) which represents the time lag between the application of shear stress and the resulting strain during the test. Figures 3-2 and 3-3 reproduced from the

Asphalt Institute Lecture Notes(31) shows the principle of operation of DSR.

The thermal cracking properties (stiffness and slope of the master curve) are measured at the anticipated lowest pavement temperature using the Bending Beam Rheometer (BBR). The BBR consists of a loading frame over which an asphalt beam made using the PAV aged binder is subjected to a mid-point loading for 120 seconds under a load of 100 grams. The Creep Stiffness of the beam is determined at varying intervals from 0 to 120 seconds and a stiffness master curve is plotted at each test temperature. The Creep Stiffness (S) and the Slope of the master curve (m) is determined at 60 seconds. Figures 3-4 and 3-5 reproduced from the Asphalt Institute Lecture Notes (31) shows the line sketch of the BBR and its principle of operation.

3.4 DETERMINATION OF PERFORMANCE GRADE OF A GIVEN BINDER USING SUPERPAVE BINDER SPECIFICATIONS

To determine the PG grade of a given binder, the Rotational Viscosity (135C) C and the flash point temperature of the unaged binder (tank asphalt) is determined. The binder is aged using the Rolling Thin Film Oven (RTFO) to simulate the aging during the mixing and laydown. The propensity of both the unaged and RTFO binder to rutting is evaluated by determining the Inverse of Loss Compliance ($G^*/\sin\delta$) at 10 rad/sec. Inverse of loss compliance measures the non-recoverable deformation of the asphalt binder when subjected to temperatures and loading rate commensurate with the traffic loading (10 rad/sec). This test fixes the higher temperature of the PG grade of the binder and is conducted at the higher anticipated pavement temperatures. Minimum values of 1.0 and 1.2 kPa have been specified for the unaged and RTFO aged binders to ensure that the mixes offer sufficient rutting resistance during mixing and lay down, and when the pavement is in service (29).

To determine the intermediate temperature below which the binder is susceptible to load-associated fatigue cracking, the Dissipated Energy ($G^*\sin\delta$) of the binder is determined using the DSR. To determine the Dissipated Energy, the RTFO aged binder is further aged in a Pressurized Aging Oven for 20 hours at 2.1 MPa (at 90 or 100 or 110 C as given in Table 3-1) to simulate the long term aging of the binder in the field. The Superpave specifies a maximum value of Dissipated Energy to be 5000 kPa at the anticipated intermediate pavement temperature (29).

To determine the lowest temperature below which the binder is susceptible to thermal cracking, the Creep Stiffness (S) of the binder and the Slope of the Stiffness Master Curve (m) at 60 seconds is used. The stiffness master curve is obtained by applying loading the PAV aged binder for 2 minutes at lowest anticipated pavement temperature. The Superpave specification allows a maximum stiffness of 300 MPa and a slope of 0.3 at 60 seconds of loading (29).

In addition to the above parameters, the Superpave binder specification specifies a minimum tensile strain at anticipated lowest pavement temperature. Figure 3-6 shows the flow chart to be followed to determine the Performance Grade of a given binder. Since the instrumentation for evaluating the tensile properties of the binders is still under critical evaluation and redesign, this parameter will not be discussed in this section.

3.5 EVALUATION OF ASPHALT-RUBBER BLENDS USING SUPERPAVE BINDER SPECIFICATIONS

Hanson et al. (32) evaluated the A-R blends prepared using 3 base asphalts, 4 CRM gradations and 5 different concentrations. They concluded that the concentration of CRM increases the stiffness of the blend at higher temperatures and decreases the same at lower temperatures. This property of CRM is said to enhance resistance to rutting, load associated fatigue cracking and thermal cracking. Hanson et. al (33) have also evaluated about 60 asphalts (both virgin and TFO aged) used throughout the United States for viscosity at 60, and 135 C, penetration at 4 and 25 C, ductility at 25 C and softening point. Their objective was to establish a correlation between the viscosity grade and their corresponding Performance Grade. They concluded that AC-5, AC-10, AC-20 and AC-40 binders would classify as PG 52-28, 58-22, 64-22, 70-16 respectively. The validity of this research was questioned by Bahia and Anderson (34) based on the wide scatter of the data

in the plots of conventional physical properties (viscosity and penetration) versus parameters like $G^{*}\sin\delta$, failure strain and creep stiffness. Bahia and Anderson (34) emphasize the need to evaluate the deformation characteristics of the binders at temperatures and loading rates that mimic the climate and traffic conditions. This is because the conventional methods to characterize the asphalt properties to pavement performance are said to not be reliable due to the empiricism involved in the determination of those properties and in their relation to the pavement performance.

McGeneiss (35) evaluated the A-R binders supplied by Rouse Rubber Industries using the Superpave test methods to conclude that:

1. Blending CRM in small quantities (7.5%) generally resulted in the PG classification being increased to one high temperature grade of the base asphalt (E.g.: from 64 to 70C), while blending moderate amounts 15% of CRM resulted in a binder classification that was generally classified two or three high temperature grades (E.g.: from 58 C to 64 or 70 C) and one low-temperature grades lower than those of the base asphalt (E.g.: -6C to -12C).
2. The RTFO may not be suitable for aging the A-R binder due to the formation of a veil of material across the bottle
3. Storing of asphalt-rubber binders over a period of time resulted in a build up of viscosity thus indicating the need to control the thermal history of the samples to obtain repeatable results.

3.6 TEST PLAN TO DETERMINE THE PG GRADE OF A-R BLENDS

In this study, it was decided to prepare CRM mixes by using three A-R blends, in addition to the evaluation of the RUMAC mixes. The A-R blends were distinguished from one another by the percentage of CRM in the binder. CRM contents of 5, 10 and 15% by weight of the asphalt cement were used to prepare the A-R blends. The blending of asphalt and CRM was accomplished using Marshall mechanical mixer with suitable modifications in terms of using temperature control on the mixing bowl to maintain the blending temperatures as recommended by the Rouse Rubber Industries (13).

After blending, about 500 grams of each of the three blends were sampled for PG grading using the Superpave binder testing instrumentation. Table 3-2 gives the amount of material used for various tests conducted to determine the Performance Grade of the A-R binders.

Table 3-2 Tests Conducted for Determining the Performance Grade of Binders

TEST TYPE	SAMPLE SIZE	AMOUNT OF BINDER USED
ORIGINAL BINDER		
Brookfield Viscosity	3	10.2 grams/sample
DSR	3	10 grams/sample
Short-Term Aging in Thin Film Oven (TFO)	4	50 grams/sample
TFO AGED BINDER		
DSR	3	10 grams/sample
Long-term Aging in Pressure Aging Vessel (PAV)	3	50 grams/sample
PAV AGED BINDER		
DSR	3	10 grams/sample
Bending Beam Test	3	15 grams/sample

3.7 PREPARATION OF ASPHALT-RUBBER BLENDS AND TESTING

The A-R blends were prepared as per the Rouse Rubber Industries recommended procedure(13). About 4000 grams of plain AC-30, which corresponds to the binder used in Unmodified and RUMAC mixes (both lab and field) was taken in a temperature-controlled deep fryer. The fryer could maintain a steady temperature of up to 232 C. The plain asphalt was constantly stirred by the Marshall mechanical whip at 160 C for about 15 minutes before addition of CRM. After 15 minutes of constant stirring, a specified amount of CRM at room temperature was added slowly and the stirring continued. The sides of the deep fryer was scrapped manually using a thin wooden scale to prevent the sticking of the CRM particles to the sides. After blending for 20 minutes, the blend was well stirred and transferred to 500 ml cans for mix preparation purposes.

3.7.1 Superpave Binder Tests on Asphalt-Rubber Blends and PG Classification

The asphalt-rubber blends prepared in the laboratory were evaluated along with the unmodified asphalt using the Superpave binder testing instrumentation at the Arkansas State

Highway and Transportation Department to obtain information about the effect of CRM on rutting resistance, fatigue and low temperature cracking. After preparing the asphalt-rubber blends, about 500 grams of the blend was transferred into sufficient number of 50 ml cans for further evaluation Superpave specifications. The binders were evaluated in five distinct stages:

1. The Brookfield viscosity was determined on three samples of each binder type in accordance with ASTM D4402 specifications to evaluate the pumpability of the binder in the field.
2. About 50 grams of the binder (both plain and A-R) was taken in a flat pan and aged in a Thin Film Oven at 163 C for 4 hours in accordance with ASTM D 1754 specifications to simulate the binder aging during the mix production and compaction. A total of six samples were aged in Thin Film Oven.
3. The Inverse of Loss Compliance ($G^*/\sin\delta$) was determined on unaged and Thin Film aged binders using the Dynamic Shear Rheometer as per the specifications.
4. Three samples of the binder aged in the thin film oven was further aged in the Pressure Aging Vessel at 100 C for 20 hours at 2.1 MPa to simulate the long term aging of the binder during its service life.
5. The PAV aged binders were evaluated for Dissipated Energy (($G^*\sin\delta$) using the Dynamic Shear Rheometer, and for the stiffness and the slope of the stiffness Master Curve using the Bending Beam Rheometer
6. Performance Grade of the binders were determined using the Superpave Binder Specifications given in Table 3-1 and as per the procedure outlined by Asphalt Institute(36). Table 3-3 shows the Performance Grade classification of the binders evaluated in this Study.

3.8 DISCUSSIONS ON PG CLASSIFICATION RESULTS

The performance grading of the unmodified and rubber modified asphalt (Table 3-3) shows that blending of crumb rubber broadened the range of applicability of the asphalt. The high temperature increased from 64 C to 80 C with 10 and 15 percent A-R rubber blends and the low temperature decreased from -22 C to - 34 C with 15 percent A-R blends. There is however, no indication of improvement in load-associated fatigue resistance.

Among the asphalt-rubber blends binders tested in this study the 15 percent A-R blend marginally (3.1 pa-s) exceeded the viscosity limits (3 Pa-s). It must be noted that Brookfield viscosity in excess of 3 Pa-s indicates that the binder could pose problems in terms of pumping during the mix production.

Table 3-3 Performance Grade Classification of the Binders Used in this Study

PG Classification Criteria	Unmodified AC-30	AR 5%[a]	AR 10%	AR 15%
Brookfield Viscosity **20 rpm, 135 C. Max 3 Pa-s**	0.42 Pa-s	0.75 Pa-s	1.66 Pa-s	3.1 Pa-s
Dynamic Shear Rheometer (Unaged) **G^*/sin(delta) kPa @ 10 rad/sec** **Temperature (C)**	64	70	80[b]	80[b]
Dynamic Shear Rheometer (TFO) **G^*/sin(delta) kPa @ 10 rad/sec** **Temperature (C)**	64	70	80[b]	80[b]
Dynamic Shear Rheometer **G^*sin(delta) kPa @ 10 rad/sec** **Temperature (C)**	25	25	25	22
Bending Beam Rheometer **Stiffness (S) MPa @ 60 Sec** **Slope of the Master Curve (m) @ 60 sec** **Temperature (C)**	-12	-18	-18	-24
PG Classification	**64 - 22**	**70 - 28**	**80 - 28**	**80 - 34**

[a]indicates % CRM by weight of asphalt cement

[b]indicates that it was not possible to test the binder in the DSR beyond 80C

The binder specifications however indicate that binders not meeting the viscosity requirements can still be considered for use in mix production if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards (29).

To summarize the results from the asphalt-rubber binder evaluation program, it can be concluded that the CRM has the potential to enhance the performance properties of the asphalt cement binder. However, it must be realized that factors like aggregate gradation and mix preparation temperatures play a significant role in translating the superior performance properties of the asphalt-rubber binder into the asphalt concrete mixes.

CHAPTER 4

EFFECT OF CRM ON MIX DESIGN PARAMETERS

To evaluate the effect of CRM on the mix design parameters, seven laboratory mixes (3 RUMAC, 3 A-R and 1 Unmodified) conforming to the Arkansas State Highway and Transportation Department's specifications[37] for Type II surface course mixes were tested. For mix design evaluation crushed aggregates were obtained from the contractor. These aggregates corresponded to five different sizes viz., passing 19 mm, passing 12.5 mm sieve, limestone screenings passing 9.5 mm (washed and unwashed), and manufactured sand. Problems with excess dust on the aggregates posed problems to the contractor in terms of achieving the desired air-voids in the mixes. Hence washed and unwashed limestone screenings were used. Asphalt cement PG 64-22 (AC-30), UltraFine GF-80 crumb rubber, and 0.5 percent of lime were used in the mixes. The CRM rubber used in the mixes had a mean particle size of 74 microns and was supplied by Rouse Rubber Industries Inc. [13]. The principal difference between the mixes evaluated in this study was in the amount of rubber used and the method used in adding it to the mix. The gradation of the individual aggregates, CRM and lime used in this study are given in Table 4-1

One mix used only the unmodified PG 64-22 binder (no rubber). The other six laboratory mixes used various percentages of rubber with three mixes having rubber added by the "wet" process (added to and blended with the asphalt cement prior to mixing with aggregate), and the other three mixes having rubber added by the "dry" process (added to the aggregates prior to mixing with asphalt cement).

Table 4.1 Gradation of Aggregates, CRM and Lime Used to Prepare the Mixes

Sieve Size (mm)	- 19.5 mm	-12.5 mm	- 6.3 mm	- 6.3 mm Washed	Sand	Lime	CRM	AHTD Specs
19.5	100	100	100	100	100	100	100	**100**
12.5	74.7	100	100	100	100	100	100	**91-100**
9.5	34.8	94.5	100	100	100	100	100	**X**
4.75	7.6	37.7	96.8	95.8	99.7	100	100	**56-70**
2.00	2.7	8.9	60.4	46	99.4	100	100	**35-43**
850μ	2.3	6.3	40.2	19.9	96.8	100	100	**26-34**
425μ	2.2	5.8	32.4	12.1	81.3	100	100	**22-30**
180μ	2.1	5.2	25.4	7.4	9.8	99.7	87.3	**9-17**
75μ	1.3	2.8	10.6	3.1	0.5	97	15	**X**

The "wet" process mixes, referred to here as "A-R" mixes, had rubber blended with asphalt in amounts of 5, 10 and 15 percent by weight of asphalt. The "dry" process mixes, referred to here as "RUMAC" mixes, had rubber mixed with the aggregates in amounts of 1, 2 and 3 percent by weight of aggregate blend.

The Job Mix Formula (JMF) for the aggregate gradations were determined for the unmodified, A-R, and RUMAC mixes by trial and error method such that they satisfied the mid-point gradation requirements for AHTD Type II surface course mixes. The final gradations for all the 7 laboratory mixes (1 unmodified, 3 A-R and 3 RUMAC) were kept the same within 1 percent variation. The aggregate gradation corresponding to the A-R mixes was the same as that used for the unmodified mixes. For the RUMAC mixes, the aggregate blend was adjusted to account for the gradation of the CRM. Table 4-2 shows the JMF for all the mixes evaluated in this study. Figure 4-1 shows the combined gradation of the aggregate or aggregate- CRM blend (mid-point gradation) used in this study.

To prepare the mixes in the laboratory for mix design and evaluation purposes, the coarse aggregates and screenings were sieved into different fractions and stored in large pans. The material passing 4.75 mm sieve was combined and used as one material. The natural sand clean from the deleterious materials was used directly in the blend preparation instead of separating them into various fractions. The amount of aggregates corresponding to each sieve size was determined using the JMF and the blend was prepared accordingly.

Table 4-2 Job Mix Formula for the Mixes Evaluated in this Study

Mix Type	% Agg. A	% Agg. B	% Agg C	% Agg D	% Sand	% Lime	% CRM	Total
Unmodified	22	21.5	24.5	16.5	15	0.5	0	100
RUMAC 1 % CRM	22	21.5	24.5	15.5	15	0.5	1.0	100
RUMAC 2% CRM	22.5	21.75	23.5	15.5	14.25	0.5	2.0	100
RUMAC 3% CRM	22.5	22	16.75	20.75	14.5	0.5	3.0	100
A-R 5%	22	21.5	24.5	16.5	15	0.5	0	100
A-R 10%	22	21.5	24.5	16.5	15	0.5	0	100
A-R 15%	22	21.5	24.5	16.5	15	0.5	0	100

4.1 PREPARATION OF CRM MIXES

The CRM examined in this study are RUMAC and A-R mixes prepared by a generic method in accordance to the specifications outlined by the Arkansas State Highway and Transportation Department. Based on the design considerations outlined in Chapter 2, it was possible to identify various standards for the preparation of CRM mixes by the dry and wet processes. Table 4-3 summarizes the standards adopted for the preparation of CRM mixes in the laboratory.

4.2 MIX DESIGN PROCEDURE

The JMF for all the 7 mixes yielded an aggregate gradation which satisfied both the AHTD Type II surface course specifications and the Superpave restricted zone (to be discussed later). After determining the JMF, the aggregates were sieved into different fractions and the weight of each fraction required for preparing an aggregate blend of 1180 grams was determined. The preparation of Marshall samples was accomplished by using the sample preparation standards established in Table 4-3. The mixing and compaction temperatures selected from viscosity considerations worked out to be 156 C and 143 C for unmodified and RUMAC mixes and 168 C and 149 C for A-R mixes. The design of unmodified mixes was accomplished using the conventional procedures outlined in Asphalt Institute MS-2 (38). For preparing the RUMAC mixes, the CRM at ambient temperature was mixed with the hot aggregates for about 15 seconds and specified amount of asphalt was added. The mixing was continued for 2 minutes using

Table 4-3 Standards for the Preparation of Fine Rubber Modified Asphalt Mixes

Details	From Literature Review	Standards Recommended
Aggregate temperature before mixing with CRM	177 C[8], 191C[25] & 218C[8]	Higher aggregate temperature is said to ensure better reaction between asphalt and CRM. However, significant benefits have not been reported by using higher mixing temperatures. Use of 177 C is recommended based on the most recently published information[12]
Duration of aggregates in the oven before dry mixing with CRM	12 hours[8]	Aggregates will be placed in the oven at 177 C for at least 12 hours before mixing.
CRM Temp before dry mixing with aggregates	AmbientTemp [1,8,23,24,25]	CRM maintained at room temperature will be mixed with the hot (177 C) aggregates.
Asphalt Temp before mixing with aggregate and CRM	135 and 149 C[8]	Asphalt will be maintained between 135 to 149 C prior to the mixing with the aggregate- CRM blend.
Mold Temp for sample prepn.	135 C[25], 160 C[23]	The mold temperature must be comparable with the mix temperature. to prevent the mix from cooling quickly. Since the aggregate batch at 149 C will be mixed with ambient CRM and asphalt at 135 C. It is possible that the temperature of the blend would be around 149 C after mixing. Use of molds maintained between 135 to 149 C is recommended.
Duration of mixing Aggregate & CRM	15 secs[8]	15 seconds of mixing time will be adopted.
Duration of mixing aggregate and CRM with asphalt.	2 Min[24], 3 Min[8]	Intimate mixing and mixing temperature of 135 and above is essential. 3 min. mixing, supplemented by heating the mixer with hot flame during mixing is recommended
Temp of compaction hammer and hot plate	149-160 C[23]	The compaction hammer face will be maintained at 149 to 160 C

Table 4-3 Standards for the Preparation of Fine Rubber Modified Asphalt Mixes (cont" d)

Details	From Literature Review	Standards Recommended
Molds treatment before adding the mix	Coat the inside of the mold with silicone grease for ease in removing the sample [8,24,25]	Dow Corning Grease will be used to coat the inner sides of the molds.
Filter paper requirements.	Use Release Paper[25,] Greased Paper[25], Greased Manila Paper[23]	Greased filter papers will be used.
Type of Compaction	50 blows[24], 75 blows[8], Gyratory[24]	75 blows will be used to be representative of the traffic conditions on I 40. Gyratory Compaction will be achieved using Superpave Gyratory Compactor at a gyratory level (Ni 8, Ndesign 86 and Nmax 152) which produces a compaction comparable with the Marshall compaction and is representative for environmental conditions typical to the State of Arkansas.
Curing	191 C[25] 219C[25], No Curing[8]	Generic mixes show increase in modulus with 2 hr. of curing. Since fine CRM is used in this study, a 2 hour curing period at 191 C is recommended.
Surcharge	2.25 Kg.[23, 25, 24]	2.2 Kg. of surcharge will be used to confined the samples with wooden plug (98 mm dia and 25 mm thick) at top and bottom. This is said to counteract swelling of the mix.
Duration of Surcharge	24 hours [8,23, 25]	Since the surcharge counteracts the swelling and that the swelling is predominant when the mix is hot, it may not be necessary to apply surcharge long after the cooling. Hence, surcharge is recommended for only 6 hours.
Sample Extrusion	After setting in the Molds overnight	6 hours or overnight is recommended, depending upon the number of mold available in the lab.

the Marshall mechanical mixer. Upon mixing, the mix was compacted in silicone greased molds by applying 75 blows on each side. After compaction, the samples were confined in the mold for 24 hours with a surcharge of 2.2 Kgs applied through a circular wooded plugs of 98 mm in diameter.

To prepare the A-R mixes, the A-R blend was first stirred thoroughly to ensure an uniform dispersion of CRM particles in the blend. The blend was then added to the hot aggregates and mixing was done for 2 minutes as in case of the conventional mixes. The Theoretical Maximum Density (TMD) of each sample was determined (ASTM D2041) at each of the four asphalt contents selected for the study. After extrusion of the samples, the bulk densities (ASTM D2726) of the samples were determined and used in the Density - Void analysis. Plots of binder content versus unit weight, air-voids, VMA, VFA, flow, and Marshall stability were generated using the results from the density-void analysis. The Optimum Asphalt Content (OAC) was determined at 4 percent air-void level and the mix properties were checked at the OAC to ensure they were within the specifications.

Previous studies (25) recommended the use of paraffin coated molds and confining the rubber modified mixes for 24 hours in the molds prior to extrusion. The product information on CRM (13) indicated that the fineness of the material would ensure quick and adequate reaction (in terms of asphalt absorption) between the CRM and the asphalt binder at the normal mixing time and reduce swelling. To evaluate the effect of mold paraffining and sample confinement on the mix design parameters, it was decided to design mixes for confined and unconfined conditions with and without paraffin coating of the molds. The design parameters of the mixes prepared for the confined and unconfined, and mold-paraffin and no mold-paraffining condition were statistically compared to evaluate the significance of sample confining and mold paraffining on mix design properties.

4.3 DESIGN OF MIXES BY SUPERPAVE VOLUMETRIC MIX DESIGN METHOD

The Superpave mix design method is the end product of the $50 million research that was performed under the Strategic Highway Research Program (SHRP). The uniqueness of the Superpave (meaning Superior Performing Asphalt Pavements) system is that the design and analysis are performed at either of three levels, (Level I or Level II or Level III) depending upon the traffic (ESALs) and environment (max. and min. pavement temperatures). The tests and data analyses are tied to the prediction of field performance. The Level I design or simply the Volumetric mix design is basically a design based on improved material selection and volumetric design procedures. Level 2 design uses volumetric design as a starting point to predict the mix performance. The Level 3 design is a more rigorous approach in which an array of tests are performed on the mixes to predict the pavement performance (39) . In this study, the design of CRM mixes was accomplished by Superpave volumetric mix design method and hence the discussions will be limited to the discussions on Superpave volumetric mix design method only.

The Superpave volumetric mix design procedure is a clear departure from conventional mix design methods like Marshall mix design method. Not only are the binders evaluated with regard to performance related parameters, the mixes are prepared in the lab to simulate field production and compaction. Two important stages in the sample preparation process of Superpave mix design are: aging of the mixes to simulate field aging, and gyratory compaction to simulate field compaction and to evaluate mix compactability for a given set of traffic and environmental conditions. Table 4-4 shows the gyratory compaction effort associated for a given traffic and environmental condition.

4.3.1 <u>Design Considerations in Superpave Volumetric Mix Design Method</u>

The Superpave volumetric mix design method accounts for the following in the design of asphalt mixes (39):

1. Selection of binders from performance based criteria
2. Selection of aggregates from consensus and source aggregate properties
3. Selection of aggregate blends from control points and restricted zone criteria (Figure 4-2)
4. Aging of the mix for 4 hours at 135 C to simulate field aging starting from mix production, storage in silos, transportation and until field compaction
5. Mix compaction using the gyratory compactor which is said to simulate the field compaction
6. Selection of compaction effort tied to climate and traffic level (Table 4-4) and
7. Selection of mix designs based on mix compactibility (Figure 4-3) and moisture sensitivity.

The volumetric mix design procedure starts with the selection of binder from performance criteria, i.e. from the maximum and minimum pavement temperature for the region where the mix is to be placed. Aggregates meeting the specifications for the consensus properties, (coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and clay content) are further evaluated for their source properties which include toughness, soundness and deleterious materials. Aggregates meeting the above properties are blended to obtain a gradation which meets the control points and restricted zone criteria. The control points in a gradation curve are those points between which the aggregate gradation must pass and the restricted zone is one between which the gradation curve must not pass. The control points are placed on the nominal maximum size, on an intermediate sieve size and on the smallest sieve size. The restricted zone lies on the maximum density gradation between an intermediate sieve size and the 0.3 mm sieve. The restricted zone criteria eliminates the use of humped gradations which are constituted by excess of fine sand in relation to the total sand. The elimination of humped gradation helps to design mixes with adequate compactibility, rutting resistance and VMA (39).

Three gradations are selected as trial gradations and the trial asphalt contents of these mixes are determined in accordance with the procedures outlined in the Asphalt Institute SP-2 Manual (39). Two samples are prepared at the trial AC content and the gradation that best meets the compactibility and VMA criteria is selected for further evaluation. The compactive efforts are selected from Table 4-4 depending upon the 7 day maximum air temperature and traffic level (39).

Two specimens are prepared at the trial asphalt content, at 0.5% above and below the trial AC content and at 1.0 percent above the estimated asphalt content. The mix properties are evaluated at the three compactibility levels referred to as $N_{initial}$, N_{design}, and $N_{maximum}$. The volumetric properties are calculated at N_{design} and plotted to determine the OAC at 4 percent air-voids. The mix properties are checked at this asphalt content to ensure that they meet the design criteria (39). If they do, then this is selected as the design asphalt content.

4.3.2 <u>CRM Mix Design by Superpave Volumetric Mix Design Method</u>

This part of the research was undertaken to determine the design asphalt content for mixes using traffic levels comparable to that assumed for the Marshall mix design and for environmental conditions typical to the State of Arkansas (design 7-day maximum air temperature less than 39 C). The objective was to develop a comparison of mix properties for mixes designed using the two procedures. At this stage, it is again emphasized that in the Superpave method, the evaluation of binder and aggregates precedes the volumetric design of the mixes. Since the main objective in this part of this study was to compare the Superpave volumetric mix design with the Marshall mix design for a given aggregate gradation, it was decided to bypass the aggregate evaluation tests and proceed directly with the volumetric design of the mixes. In the Superpave mix design, the maximum number of gyrations to which the mixes are compacted depends upon the traffic and environmental conditions (39). The design number of gyrations (N_{design}) comparable to the traffic conditions used in Marshall procedure and satisfying the Arkansas environmental criteria was 96. Corresponding values for the initial ($N_{Initial}$) and maximum (N_{max}) number of gyrations were 8 and 152 respectively.

The JMF of the aggregate blend used in the Superpave volumetric mix design were kept the same as that used in the Marshall mix design. Two replicates were prepared at each asphalt content at a gyratory compaction level of N_{max} = 152 gyrations. The mixing temperature was the same as used in the Marshall method. However, the mixes were aged for 4 hours at 135 C and compacted at 150 C.

Eight kilogram aggregate batches were used in the Superpave volumetric mix design. About 6.5 kilograms of the mix were used to prepare test specimens of 150 mm diameter and 150 mm in height. Two samples were prepared at each binder content using the mixing and compaction temperature adopted in the Marshall mix design procedure. The mixes were aged for 4 hours at 135 C, brought to appropriate compaction temperature, and compacted at a maximum gyratory compaction effort of 152 gyrations.

The bulk specific gravity (BSG) of the samples were determined (ASTM D 2726) after the samples cooled to the room temperature. The data acquired during the mix compaction were retrieved into a spreadsheet to compute the mix density at each gyration. Using the BSG and the TMD (ASTM D2041), a correction factor was derived and the densities at all the gyrations were corrected. The percent compaction at N_i = 8, N_{design} = 96 and N_{max} = 152 were compared with the Superpave specifications. If the mix satisfied the compactibility conditions at $N_{initial}$ and $N_{maximum}$ gyratory compactive effort, then a volumetric analysis was performed to develop plots of air-voids, VMA and VFA with the varying binder content. The optimum asphalt content (OAC) was determined at 4 percent air-voids level and the mix properties were checked at the OAC to ensure that they met the specifications.

4.4 DISCUSSION OF THE MIX DESIGN RESULTS

4.4.1 Discussions on Marshall Mix Design Results

1. Table 4-5 lists the Marshall mix design results. These mix design results for the laboratory mixes indicate that for the "dry" process, the GF-80 crumb rubber added at 1 and 2 percent CRM had no significant effect on the OAC, VMA or VFA; however, stability decreased with increasing rubber percentages (17124 N unmodified, 15034 N at 1 percent, and 9875 N at 2 percent). With 3 percent CRM the OAC increased from 5.1 to 5.7 percent, VMA increased (15.5 to 16.2 percent), VFA decreased (73 to 65 percent), and the Marshall stability continued to decrease (7828 N). It can be seen that the effect of CRM on the OAC and volumetric properties is significant for RUMAC mixes with 3% CRM. This expected behavior of the "dry" process mixes could be attributed to the absorption of asphalt by the CRM which increases the asphalt content requirements for the mix to attain the required volumetric properties in the mixes (in this case, the air voids).

Table 4-5 Marshall Mix Design Results for Unmodified, Rubber Modified and Asphalt-Rubber Mixes

		LAB - RUMAC MIXES			LAB A-R MIXES		
Design Parameters	**Unmod**	**1%* CRM**	**2%* CRM**	**3%* CRM**	**5%** A-R**	**10%** A-R**	**15%** A-R**
OAC %	5.1	5.1	5.1	5.7	5.2	5.6	5.8
VMA (%) Min. 15.2%	15.5	15.4	15.1	16.2	15.8	16.3	16.6
VFA (%) Range 65-75%	73	74.0	74.0	65.0	72	76	79
Stability (N) Min 8000 N	17124	15034	9785	7828	19793	18904	18503
Sp. Gr. of Binder	1.033	1.033	1.033	1.033	1.043	1.047	1.051

* Percentage of CRM in the mix expressed as the total weight of the aggregate blend
**Percentage of CRM in the A-R Blend expressed as a total weight of the asphalt cement binder

2. Although an increase in CRM content in RUMAC mixes did not significantly affect the resulting OAC, similar trends were not observed in case of A-R mixes designed using A-R blends having varying percentages of CRM content. This could related to the benefits of blending asphalt and rubber prior to mixing with the aggregates, a process which ensures adequate reaction between the two materials. Hence, it can be seen that the OAC of the A-R mixes are less affected by the absorption of asphalt by the CRM.
3. The addition of crumb rubber by dry process seems to reduce the stiffness of the mixes, as indicated by a reduction in the Marshall stability. The decrease in Marshall stability with an increase in the percentage of CRM in dry-process mixes may be an indication that 2 minutes of mixing and limited aging of the mix does not permit adequate reaction (in terms of asphalt absorption) between the asphalt and rubber to produce a modified blend, as proposed [13] by the CRM producer.

4.4.2 Effect of Sample Confinement and Paraffin Coated Molds

From Tables 4-6 it can be seen that the CRM mix samples prepared for sample confining and sample unconfined conditions do not show distinct differences in terms of the mix design parameters. Tests for hypothesis indicated no significant differences between the mix design parameters of the CRM mixes designed for either confined vs. unconfined

Table 4-6 Marshall Mix Design Parameters for RUMAC Mixes for Various Paraffining and Sample Confining Conditions

Mix Type	Condition	OAC %	%VMA (Min 15.2%)	%VFA 65-75%	Stability Min 8000 N	Flow (2- 4 mm)
Unmodified	**No Paraffin Unconfined**	5.15	15.5	75	17124	2.75
RUMAC 1% CRM	**No Paraffin Confined**	5.1	15.2	74	14223	3.0
RUMAC 1% CRM	**No Paraffin Unconfined**	5.1	15.2	75	15034	3.0
RUMAC 1% CRM	**Paraffin Confined**	5.15	15.3	75	12632	2.75
RUMAC 1% CRM	**Paraffin Unconfined**	5.1	15.1	74	12854	2.75
RUMAC 2% CRM	**No Paraffin Confined**	5.05	15.1	76	10141	2.75
RUMAC 2% CRM	**No Paraffin Unconfined**	5.1	15.1	76	9785	3.0
RUMAC 2% CRM	**Paraffin Unconfined**	5.1	15.1	76	9385	2.75
RUMAC 3% CRM	**No Paraffin Confined**	5.6	16.1	76	8406	4
RUMAC 3% CRM	**No Paraffin Unconfined**	5.7	16.2	76	7828	3.9

samples or for the paraffined vs. non-paraffined mold conditions. Based on this evidence, subsequent mix designs were performed without using confinement and without paraffin-coated molds. The results from the student "t" test for significance is shown in Table 4-7.

4.4.3 Discussions on Superpave Volumetric Mix Design Results

1. From Tables 4-8 to 4-10 can be seen that for the aggregate, crumb rubber type and the aggregate gradation used in this study, the Superpave volumetric mix design procedure yields a lower OAC than the Marshall method. The reduction in the OAC (between the Marshall and the Superpave procedures) ranges from 1.0 to 1.3 percent for the dry process and 0.8 to 1.1 percent for the wet process. It is recognized that none of the Superpave mixes met the VMA criteria, and therefore are not acceptable mixes. This is a result of the fact that the aggregate gradation was held fixed at values selected from the AHTD Specifications. However, this does not invalidate the conclusion that for a fixed gradation and aggregate blend the Superpave volumetric mix design procedure produces a lower OAC.
2. Table 4-9 and 4-10 shows mix design results for both Marshall mix design and the Superpave volumetric mix design procedure. When comparing the specimens fabricated during the respective mix design processes, it is apparent that the specimens exhibit different volumetric properties. The Superpave volumetric mix designs resulted in a lower optimum asphalt content, VMA and VFA relative to the

Table 4-7 Statistical Analysis Showing the Effect of Sample Confining and Paraffining on the VMA of RUMAC Mixes at 5.5% Asphalt Content

Mix I.D	Effect Evaluated	Sample Size	Mean VMA %	Std. Dev.	t)Cal	t)5%	Remarks
RUMAC 1%	**Confinement**						
No Paraffining	**Unconfined**	3	15.03	0.13	0.42	2.78	Not Significant
	Confined	3	15.07	0.10			
RUMAC 2%	**Confinement**						
No Paraffining	**Unconfined**	3	15.02	0.23			
	Confined	3	15.02	0.056	0.21	2.78	Not Significant
RUMAC 3%	**Confinement**						
	Confined	3	16.45	0.06		2.78	Not Significant
	Unconfined	3	16.45	0.10	0		
RUMAC 1%	**Paraffining**						
	Paraffin	3	15.1	0.12		2.78	Not Significant
	No Paraffin	3	15.1	0.10	1.1		
	Paraffining						
RUMAC 2%	**Paraffin**	3	15.1	0.15		2.78	Not Significant
	No Paraffin	3	15.1	0.13	0		

Table 4-8 Superpave Volumetric Mix Design Results for Unmodified, RUMAC and A-R Mixes

Mix Design Parameters	Unmodified Mix	RUMAC Mixes (Dry - Process)			A-R Mixes (Wet - Process)		
		1% CRM	2% CRM	3% CRM	5% A-R	10% A-R	15% A-R
OAC (%)	4.1	4.1	4.4	4.4	4.4	4.7	4.7
VMA (%)	11.5	11.4	13.0	11.2	12.1	13.9	13.2
VFA (%)	65	65	70	72	65	70	68
% Compaction @ $N_{initial}$ (< 89%)	88.7	88.5	88.7	88.9	88.3	88.6	88.6
% Compaction @ N_{max} (<98%)	97.9	97.6	97.8	97.8	97.2	97.4	97.5

[a]Percentage of CRM in the mix expressed as the total weight of the aggregate blend
[b]Percentage of CRM in A-R Blend expressed as total weight of asphalt cement binder

Table 4-9 Comparison of Marshall and Superpave Volumetric Mix Designs for Unmodified and RUMAC Mixes

	Unmodified Mixes		RUMAC 1%[a]		RUMAC 2%		RUMAC 3%	
Mix Parameters	Marshall	Superpave	Marshall	Superpave	Marshall	Superpave	Marshall	Superpave
OAC %	5.1	4.1	5.1	4.1	5.1	4.1	5.7	4.4
VMA (%) Min. 15.2%	15.5	11.5	15.4	11.4	15.1	13.0	16.2	11.2
VFA (%) Min. 65% Max. 75%	73	65	74	65	74	70	65	72

[a]Percentage of CRM expressed as the total weight of the aggregates

Table 4-10 Comparison of Marshall and Superpave Volumetric Mix Designs for Unmodified and A-R Mixes

	Unmodified Mixes		**A-R 5%[a]**		**A-R 10%**		**A-R 15%**	
Mix Parameters	**Marshall**	**Superpave**	**Marshall**	**Superpave**	**Marshall**	**Superpave**	**Marshall**	**Superpave**
OAC %	5.1	4.1	5.2	4.4	5.6	4.7	5.8	4.7
VMA (%) Min. 15.2%	15.5	11.5	15.8	12.1	16.3	13.9	16.6	13.2
VFA (%) Min. 65% Max. 75%	73	65	72	65	76	70	79	68

[a]Percentage of CRM expressed as the total weight of the asphalt cement binder

Marshall mix design procedure, for unmodified mixes and all rubber-modified mixes. This trend in volumetric data agrees with D'Angelo, et. al (40) for mixes compacted using the same N_{design} and Marshall compactive effort.

A possible reason for the discrepancy in the volumetric data could be a reduction in the effective asphalt content of the Superpave mixes due to asphalt absorption by the aggregates and crumb rubber during the aging process within the Superpave procedure. A study by Hafez and Witzack in which unmodified and rubber-modified mixes designed using the Marshall method were aged for 1 hour at 160 C prior to compaction did not report consistent differences in the optimum asphalt content between the Marshall specimens and Superpave specimens (41). However, the differences in duration of mix aging -- no aging under conventional Marshall procedures vs. 4 hours at 135 C under Superpave procedures ‟ could be a major factor in differences in observed volumetric data.

Another possible explanation for the discrepancy in the volumetric data between the Marshall and Superpave specimens is that the relative compactive efforts are not in fact comparable. The basic premise of the relative compactive efforts is the same, namely, the compactive efforts result in specimen densities expected after pavement has been "in-service" for some period of time. The Marshall mix design was performed using the compactive effort (75 blow per side) for "heavy" traffic ($>10^6$) ESAL). The Superpave volumetric mix design was performed using a compactive effort (N_{design} = 96; $<10^7$ ESAL), meant to be comparable to the Marshall effort, in terms of design traffic level. However, there was no information available to correlate the actual compactive effort generated by the gyratory compactor to that generated by the Marshall hammer. To generate such a correlation between the gyratory compactor and the Marshall hammer was beyond the scope of this study.

CHAPTER 5
EVALUATION OF CRM MIXES FOR PERFORMANCE

One of the primary objectives of this study was to evaluate the effect of adding crumb rubber to asphalt mixes. A major tool for this evaluation is performance related properties. Testing was performed to show the effect of increasing amounts of crumb rubber on these properties. The mixes tested were designed using both Marshall and Superpave volumetric mix design procedures. The mixes were kept as consistent as possible (identical aggregate gradation, asphalt cement type, amount of rubber additive) to facilitate meaningful comparisons, both within the mix design types and between the mix design types. However, the volumetric properties between mix design types (Marshall versus Superpave) are not similar. In fact, the Superpave mixes do not meet current AHTD or Superpave volumetric specifications. Thus, comparisons of performance related data between Superpave and Marshall mixes in this study are meaningless. However, observations of the trends in performance related properties within a particular mix design type can shed light on the effect of increasing rubber content on the properties of the mix. Therefore comparisons are given for Marshall-designed mixes and for Superpave-designed mixes.

The evaluation of the performance properties of CRM mixes was a major phase of this research study. The CRM mixes were critically evaluated for their performance from several considerations in addition to the original plans outlined in the research project proposal. To evaluate the CRM mixes for performance properties, the samples were prepared for the following criteria:

a. Lab Marshall Samples: These samples correspond to the laboratory mixes designed at the University of Arkansas, Fayetteville using the Marshall method in accordance with the Asphalt Institute's MS -2 manual (38). The aggregates, AC and CRM used in these designs were procured from the field contractor.

b. Lab Superpave Volumetric Mix Design Samples: These samples correspond to the laboratory mixes designed at the University of Arkansas, Fayetteville (UAF), using the Superpave volumetric mix design method based on the procedure outlined in the Asphalt Institute SP-2 manual (39). The mixes were aged for 4 hours at 135 C prior to compaction by the SGC. As a result, the design asphalt content of these mixes differ from the asphalt content of the Marshall Mixes.

It is again emphasized here that none of the Superpave mixes meet the VMA criteria and hence are not acceptable mixes. These mixes are being evaluated for performance properties to determine the effect of CRM on mixes with varying amounts of CRM.

d. Field Beam Samples : These samples were taken from the the RUMAC overlays placed on Interstate-40. The field beam samples had a CRM content of 1.0, 1.5 and 2.0% and were evaluated for their fatigue characteristics only. The design of field mixes were accomplished by the construction contractor and the mixes had a design asphalt content of 5.1, 5.6 and 5.8% respectively.

The above mentioned laboratory samples were of two types, namely, the "dry process" RUMAC mixes prepared using 1, 2 and 3% CRM, and the "wet process" A-R mixes prepared using 5, 10 and 15% A-R blends. Both types of mixes were prepared using the job mix formula corresponding to the UAF mix designs.

Although samples were prepared using different criteria during the laboratory studies, a basis had to be established to compare the test results. Six Marshall sized samples (100 mm dia and 62.5 mm height) of each mix type (RUMAC and A-R) prepared using Marshall compaction (for Marshall mixes) and Superpave gyratory compaction (for Superpave Mixes) at their respective optimum asphalt content were used for performance evaluation studies. Since

Superpave Level II and III performance test procedures and equipment are still being evaluated and refined, it was decided to evaluate the two mix designs using more traditional tests like the Repeated Load Dynamic Compression, Resilient Modulus (ASTM D 4123) and Indirect Tensile Strength tests. The fatigue characteristics of the CRM mixes were evaluated using cantilever type of loading using a test setup which was fabricated solely for this study.

At this stage, it must be noted that as the Marshall and the Superpave gyratory compacted samples were not of the same dimensions, the difference between the sizes of traditional Marshall and Superpave specimens was resolved by sawing and coring the Superpave gyratory compacted specimens. Gyratory compacted samples (150 mm dia and 150 mm height) were sawed into two samples of 62.5 mm in height, each of which were cored to a diameter of 100 mm. Thus one gyratory compacted sample (150 mm height and 150 mm dia) produced two Marshall-sized samples (100 mm dia and 62.5 mm in height). Six samples prepared at Marshall and Superpave OAC were tested for the performance-related tests previously listed.

5.1 EVALUATION OF THE RUTTING RESISTANCE OF CRM MIXES

Rutting is a flexible pavement distress caused by the accumulation of permanent deformation in the pavement layers from the repeated application of traffic. Excessive rutting in asphalt pavements is a major concern among the highway engineers. Lister and Addis (42) indicate that a rut depth in excess of 10 mm could result in the loss of structural strength and those in excess of 12.5 mm (for pavements having a cross slope of 2.5 percent) could result in ponding. Ponding creates a potential safety hazard since it can lead to wet weather skidding accidents i.e., hydroplaning and steering problems (43). Though premature failure of the pavements due to rutting can be mainly attributed to the repeated application of heavy axle loads operating at tire pressures as high as 725 kPa, the aggregate, binder and environmental factors also contribute to rutting (42,43,44).

The current trend in the highway construction is with the experimentation of CRM in asphalt mixes. Researchers (1,2) claim that incorporation of CRM into asphalt mixes will make the mixes more elastic at higher service temperatures thus enhancing their rutting resistance. This emphasizes the need to evaluate the rutting resistance of asphalt mixes through reliable test methods.

Dawley et al. (44) have classified different types of rutting as wear rutting, structural rutting and instability rutting. Wear rutting is caused by environmental and traffic influences which result in the progressive loss of coated aggregate particles from the pavement surface. The rate of wear rutting has been found to accelerate in the presence of ice-control abrasives. Structural rutting is due to permanent vertical deformation of the pavement structure under repeated traffic under repeated traffic loads. This type of rutting is usually a reflection of the permanent deformation within the subgrade. Instability rutting is caused due to the lateral displacement of material within the pavement system and occurs predominantly on the wheel paths. Instability rutting occurs when structural properties of the pavement layers are inadequate. Figure 5-1 shows the different types of rutting. Based on the above definitions, it can be inferred that this research study confines itself to the evaluation of the conventional and CRM mixes to structural rutting.

Rutting in asphalt mixes, which predominantly occurs during high temperature seasons, is affected by external factors such as pavement geometry, axle loads, contact pressure, surface shear stresses, and the bonding between the pavement layers. Shatnawi (45) quotes Kennedy (46) as indicating that rutting within an asphalt mix is controlled by the aggregates, aggregate gradation, type and amount of mineral filler, binder content, and the Voids in Mineral aggregates (VMA). The discussion on all the individual factors affecting the rutting resistance of the mixes is beyond the scope of this study. However, the effect of factors relevant to this study viz., aggregate gradation, size, shape, binder type, asphalt mix properties and additives on rutting has been summarized in Table 5-1 (43).

Researchers have evaluated CRM mixes for rutting resistance through laboratory studies and field evaluation. Laboratory evaluation of samples from field projects in Virginia (18) indicated that the use of CRM in asphalt mixes by the wet process may not enhance the rutting resistance of the mixes. Maupin (18) cautions that their laboratory tests may have not simulated the pavement deformation behavior adequately. Krutz and Stroup-Gardiner (47) on the other hand indicate that the incorporation of CRM by the dry process does enhance the rutting resistance of the mixes at higher temperatures. Similarly, Rebala et. al (48) indicate that mixes designed using 10 percent CRM and the TxDOT CRM mix design procedure produced rut resistant mixes; however, they add that the use of CRM in the dry process allows the CRM to serve as discrete particles which may enhance the rutting resistance but intensify the propensity of the mix to cracking. Initial evaluation of CRM mixes placed on the NJDOT projects indicated that rutting in CRM sections were similar to that in conventional sections. Hanson et. al (49) evaluated the field cores taken from a CRM mix test section in Columbus, Mississippi, along with the laboratory samples prepared using the field mixes. They concluded that the field compacted control mixes deformed more than the field compacted CRM mixes. However, the lab compacted samples of the control and CRM mixes did not show any significant difference in their rutting resistance. The evaluation of field projects indicated that after 2 years, the amount of rutting in the control and the CRM sections were insignificant. In short, there is no clear indication on consensus from previous researchers on whether or not CRM is beneficial relative to rutting resistance.

5.2 RUTTING RESISTANCE STUDIES

In this study, the rutting resistance of the mixes was evaluated using the repeated load dynamic compression test. The MTS or the "Material Testing System" was used in this research program to conduct the tests. This test uses the permanent undergone by the test specimens at 10,000 load repetitions as a measure of rutting resistance. Table 5-2 shows the testing matrix adopted to evaluate the rutting resistance of the mixes.

Table 5-2 Testing Matrix for Rutting Resistance Tests at 40 C

Mix Type	Marshall	Superpave
Unmodified	3	3
RUMAC 1%[a]	3	3
RUMAC 2%	3	3
RUMAC 3%	3	3
A-R 5%[b] CRM	3	3
A-R 10% CRM	3	3
A-R 15% CRM	3	3

[a]Percentage of CRM expressed as the total weight of the aggregate blend
[b]Percentage of CRM expressed as the total weight of asphalt cement
Total Number of Rutting Resistance Tests Conducted: 42

5.2.1 The MTS

The MTS is a sophisticated equipment which uses the "Closed Loop", servo control hydraulic testing system to apply dynamic loads to the test specimen. This system has the capability of applying loads on the test specimens in a manner to simulate the field conditions. The data acquisition is done by a computer interfaced with the testing unit. Figure 5-2 shows the MTS. The timing of the dynamic loads is selected in such a way as to simulate the "actual load" pulses on the pavements by the vehicles. The seating and dynamic stress maintained during the test was 3.4 kPa and 103.4 kPa respectively. The dynamic stress was reached in 0.02 sec, was maintained for 0.06 seconds, and relieved in 0.02 sec. In other words, the loading was applied in a time frame of 0.1 seconds. The load was repeated after a rest period of 1.9 seconds for a cycle time of 2.0 seconds. Figure 5-3 shows the representation of the loading sequence on the test specimen.

The tests were conducted in an environmental chamber placed on the MTS test frame . The area of the test chamber was of sufficient size to accommodate test specimens awaiting testing. The temperature inside the chamber was maintained at 40 C using a heat tape connected to a thermostat.

The load applied to the test specimen was measured using a load cell and the deformations undergone by the test specimen was measured by the strain gauge attached to the test specimen. The test data which include repetition count number, measured load, and peak and valley deformations were recorded by the computer interfaced with the test equipment. The reporting interval was maintained as 60 seconds throughout the experiment. The analysis of the data was performed by retrieving the data into a spreadsheet.

5.2.2 Test Procedure for Repeated Load Dynamic Compression Tests

The electronics (i.e., the load, strain sensitivity, loading sequence) were set and the environmental chamber was installed on the platform of the MTS. The heat tape was attached in the chamber and the electrical connections were made with the temperature controller to maintain a temperature of 40C. The hydraulic system was turned on and the machine was warmed for 20 minutes before beginning the test. In the meantime, the test specimen was prepared for testing by applying silicone grease and graphite powder on its top and bottom surfaces. The strain gauge was attached to the sample (on the bumper pads) using rubber bands. A 100 mm diameter steel circular plate was placed on the top of the specimens and the arrangement was transferred to the environmental chamber maintained at 40C. It may be noted that the specimens were stored in the environmental chamber at 40C for 24 hours before testing.

The "SET POINT" controller was operated to bring the loading piston onto the specimen. The loads from the piston was transferred to the specimen through a steel ball placed at the center of the steel circular plate. After setting the seating load to 3.4 kPa, the computer program was activated. The data acquisition and the application of the repeated dynamic loads were started simultaneously. The "DISPLAY" mode was used to set the dynamic loads to 103.4 kPa. Since each load was repeated every 2 seconds (duration 0.1 second), each experiment took about 5.5 hours. The data obtained was saved before exiting the program. With prior planning, it was possible to test three, and sometimes even four specimens in a day.

5.2.3 Analysis of the Rutting Resistance Test Data

The rutting potential of the mixes was determined from the permanent strain accumulated by the test specimens at the end of 10,000 load repetitions. The first 60 load repetitions are considered to condition the test specimen by minimizing the effect of minor specimen surface irregularities. The permanent strain was calculated as the ratio of the accumulated permanent deformation after 10,000 load repetitions to the gage length of the strain gauge (i.e., 50 mm).

To analyze the test data and make statistically relevant conclusions about the rutting resistance of the CRM mixes, a One Factor Analysis of Variance (ANOVA) test was performed using the Statistical Analysis Software (SAS) package (50). The one factor ANOVA test indicated the role of mix type on the rutting resistance of the Unmodified and RUMAC mixes, and Unmodified vs. A-R mixes.

A SAS program written for this purpose provided information in terms of the probability ($Pr > F$) that the effect of mix type on permanent strain (rutting resistance) of the unmodified and the RUMAC mixes (or the Unmodified and A-R mixes) being significant. Probability values greater than 5% indicated that the rutting resistance (permanent strain) of the mixes did not differ significantly. The statistical analysis was further extended to determine the Least Significant Difference (LSD) in the mean permanent strain of a pair of mixes. Any two mixes (from a given set) having a difference in permanent strain less than the LSD are considered not significantly different. The LSD was determined using the relation

$$LSD = t_{a/2} \text{ SQRT } [\ 2MSE/\ n\ ] \text{ ....5-1}$$

where,

LSD	=	Least significant difference in means
$t_{a/2}$	=	Student "t" value for a degree of freedom (n-k)
k	=	Number of mixes
a	=	Type I error probability (5% in this case)
MSE	=	Mean square error (obtained from SAS output)

Appendix A shows the SAS Program and a sample output from the ANOVA test.

5.2.4 Rutting Characteristics of the CRM Mixes Evaluated in this Study

Table 5-3 shows the results from the one factor analysis of variance test. From Table 5-3 it can be seen that in this study, the mix type has a significant effect on the rutting resistance. The mix sets considered in the one factor ANOVA were Marshall - Unmod & RUMAC, Marshall - Unmod & A-R, Superpave - Unmod & RUMAC, and Superpave - Unmod & A-R mixes. In each case the difference in measured rutting resistance was found to be statistically significant.

Table 5-3 Summary of One Factor ANOVA Test on the Rutting Resistance Data

Mix Combination	**Probability Associated with ANOVA Test**	**Remarks**
Unmodified and RUMAC Mixes - Marshall Design	0.0001	Mix Effect on Rutting Resistance significant
Unmodified and A-R Mixes Marshall Design	0.0001	Mix Effect on Rutting Resistance significant
Unmodified and RUMAC Mixes - Superpave Volumetric Design	0.0001	Mix Effect on Rutting Resistance significant
Unmodified and A-R Mixes Superpave Volumetric Design	0.0001	Mix Effect on Rutting Resistance significant

Table 5-4 gives the summary of the results from the statistical analysis which was extended to determine the least significant difference in the mean rutting resistance of the mixes. The general comments on the rutting resistance test results of the CRM mixes are:

a. The Marshall unmodified mix shows less permanent strain when compared to the Marshall RUMAC mixes. Among the Superpave mixes, the Superpave unmodified mix shows the highest permanent strain when compared to other Superpave- CRM modified mixes.

b. Both Marshall and Superpave RUMAC mixes show an increase in permanent strain with an increase in the percent crumb rubber in the mix.

c. The A-R mixes designed by Marshall mix design method showed an increase in rutting resistance (i.e. reduction in permanent strain) with an increase in the percent CRM in the blend. Among the Superpave A-R mixes, there was no significant difference between the rutting resistance of A-R 5% and A-R 10% mixes. However the rutting resistance of the A-R 15% mix was significantly lower when compared to those of A-R 5% and A-R 10% mixes.

d. A general trend about the behavior of Marshall mixes is that the dry process of incorporating CRM into asphalt mixes reduced the rutting resistance of the resulting RUMAC mixes while the wet process of incorporating CRM into the mixes enhanced the rutting resistance of the resulting A-R mixes. This trend was true for only the Marshall mixes which satisfied the AHTD mix design criteria.

Table 5-4 Least Significant Difference (LSD) in Mean Permanent Strain of the Mixes Evaluated in this Study

	OAC (%)	Marshall Mixes (SETI)		OAC (%)	SUPERPAVE Level I Mixes (SET III)
Mix Type		Mean Strain (mm/mm)	**Mix Type**		Mean Strain (mm/mm)
Unmod	5.1	0.020[a]	**Unmod**	4.1	0.254[c]
RUMAC 1%	5.1	0.027[ab]	**RUMAC 1%**	4.1	0.048[a]
RUMAC 2%	5.1	0.034[b]	**RUMAC 2%**	4.1	0.045[a]
RUMAC 3%	5.7	0.056[c]	**RUMAC 3%**	4.4	0.057[b]
LSD (mm/mm)		***0.008***			***0.006***
		Marshall Mixes (SET II)			**SUPERPAVE Level I Mixes (SET IV)**
Mix Type		Mean Strain (mm/mm)	**Mix Type**		Mean Strain (mm/mm)
Unmod	5.1	0.020[a]	**Unmod**	4.1	0.254[c]
A-R 5%	5.2	0.046[b]	**A-R 5%**	4.4	0.020[a]
A-R 10%	5.6	0.022[a]	**A-R 10%**	4.7	0.019[a]
A-R 15%	5.8	0.018[a]	**A-R 15%**	4.7	0.034[b]
LSD (mm/mm)		***0.010***	**LSD** mm/mm)		***0.006***

Means in the same set followed by the same letter are not significantly different at $a = 0.05$.

e. From Table 5-4 it can be seen that the Superpave-unmodified mix has undergone excessive permanent strain when compared with the RUMAC and A-R mixes. At the outset, this observation leads to a conclusion that the rutting resistance test results for the Superpave-unmodified mix is an outlier rather than a true representation.

It should be noted here that the excessive permanent strain undergone by the unmodified mixes could be tied to inadequate binder content in the unmodified mix (4.1% OAC) to coat the aggregates completely. Such a mix deficient in asphalt content cannot bind the aggregates into a matrix to adequately resist the compressive and shear stresses as applied during the repeated load dynamic compression test. The absence of similar trends in the Superpave - CRM mixes (having similar low OAC when compared to the Marshall - CRM mixes) leads to a conclusion that aging of the CRM mixes during Superpave mix preparation processes could have caused adequate Asphalt-CRM reaction to impart superior properties to the CRM mixes in terms of rutting resistance.

f. In the statistical analysis, the Least Square Difference in Means (LSD) provides a tool to identify the mixes whose permanent strain (rutting resistance) do not differ at 5% level of significance. Table 5-4 indicates that the rutting resistance of Marshall - Unmodified

(OAC 5.1%) and 1% CRM (OAC 5.1%), and Marshall 1% RUMAC (OAC 5.1%) and RUMAC 2% (OAC 5.1%) mixes do not differ significantly in their rutting resistance. Since the above mixes have the same asphalt content in them, the trend thus obtained leads to a conclusion that the dry process of incorporating the CRM into asphalt mixes may not permit the necessary asphalt-rubber interaction to affect the rutting resistance of the mixes. Similar trends can also be seen for Superpave - RUMAC 1 and 2% mixes. Although this was the case with the Marshall -Unmodified, RUMAC 1 and 2% CRM mixes, there was no significant difference between the rutting resistance of the Marshall -RUMAC 2% and 3% mixes even though the mixes differed significantly in their optimum asphalt content (5.1 and 5.7%) and no explanation could be offered for this behavior of the mixes.

To summarize, the incorporation of CRM into the asphalt mixes by the "dry" process did not enhance the rutting resistance of the RUMAC mixes as evaluated using the repeated load dynamic compression tests. Improvements in rutting resistance (as measured by repeated load test) were observed only for the Marshall A-R mixes which satisfied the mix design specifications.

5.3 EVALUATION OF RESILIENT CHARACTERISTICS OF CRM MIXES

Resilient modulus is defined as the ratio of the repeated stress to the corresponding resilient strain. Since the recoverable portion of the strain is measured in a resilient modulus test, this stiffness of the material can be related to the modulus of elasticity of the asphalt mix and is commonly used for mechanistic analysis (51). To determine the relative benefit of using CRM in asphalt mixes and to establish recommendations for design procedure modifications, mechanistic pavement analyses will be needed. These analyses must reflect typical Arkansas pavements and conditions and must evaluate the normal seasonal temperature ranges and their effects. To accomplish this, the relative effects of using CRM on resilient modulus of the mixes at different temperatures will be needed.

5.3.1 Factors Affecting Resilient Modulus

The most important factors that influence the resilient modulus are temperature, frequency of loading, asphalt consistency and air-voids. Shatnawi (45) quotes Bonaquist (52) that lower temperatures, higher rates of loading and higher viscosity asphalt can result in higher resilient moduli. The resilient modulus reportedly (52) increased two fold with an increase in frequency from 1 to 16 Hz. Also, for a given AC content, the resilient modulus is reported to increase with a decrease in air voids.

From resilient modulus test data, it is possible to determine the total strain, total recovered strain, and the instantaneous strain. Using these strain components, the total modulus, total resilient modulus and instantaneous resilient modulus are computed. Although an increase in the total number of load repetitions is said to increase the strain and reduce the resilient modulus (51), Vallejo et al. (53) have evaluated the effects of repeated indirect tensile stress on strain, modulus of elasticity and Poisson ratio. They concluded that an approximately linear relationship exists between the total resilient strain and the number of load repetitions, up to about 60 to 70 percent of the fracture life. Beyond this stage, the resilient strain increases more rapidly until failure or fracture of the sample. Figure 5-4 shows the effect of repeated loads on total resilient tensile strain. The salient features of Figure 5-4 are:

Zone of initial adjustment to the load, which consists of the first 10 percent of the fracture life. A slight curvature is exhibited in this zone indicates that the specimen is probably adjusting to load and undergoing some additional compaction.

Zone of stable condition, which is generally between 10 to 70 percent of the fracture life of the mix. In this zone, the permanent strain exhibits a linear relationship with the number of load repetitions. This zone represents the useful life of the specimen with respect to the pavement rutting.

Failure Zone, which extends from 70 percent of the fracture life to the instant of complete fracture. This zone also corresponds to the zone of excessive resilient strain in which the specimen experiences all forms of load associated distress.

5.3.2 Measurement of Resilient Modulus

Resilient Modulus is measured by using a test device such as the Retsina apparatus shown in Figure 5-5. The test samples (typically 100 mm diameter and 62.5 mm in height), are loaded on diametral axis and the deformation created along the horizontal axis is measured. This test is known as the Diametral Resilient Modulus Test. The tensile properties thus determined are referred to as the indirect tensile properties because a direct tensile force is not applied to produce the stresses in the horizontal and the vertical directions. Figure 5-6 shows the stress distribution along the vertical axis. Figure 5-7 shows a typical load deformation plot for two cycles using the Retsina equipment. The deformations are recorded at 0.1 sec after the start of each load pulse. When loads are applied pneumatically, the time at which the load peaks and the shape of the load versus time plot can vary with the size of pneumatic load applicator. Two different devices may produce a slightly different data. Therefore, a load versus time plot similar to Figure 5-7 must be determined for each test apparatus.

The resilient modulus based on the horizontal and vertical deformation can be determined using the equations given below (51). These equations are for a 100 mm diameter specimen with 1.3 cm loading strip.

$$\text{Maximum Tensile Stress} = \frac{0.156\ P}{t} \qquad \text{...............................5-2}$$

$$\text{Maximum Compressive Stress} = \frac{0.475\ P}{t} \qquad \text{............................... 5-3}$$

$$\text{Diametral Modulus } (M_d) = \frac{P\ (\mu + 0.2734)}{(t)\ (H_t)} \qquad \text{................................ 5-4}$$

$$\text{Poisson Ratio } (\mu) = \frac{3.59\ (H_t)}{V_t} - 0.270 \qquad \text{............................. 5-5}$$

Where
P = Load applied (N)
t = Specimen thickness (mm)
H_t = Total horizontal deformation (mm)

5.3.3 Limitations of Resilient Modulus Testing System and the Equations

Stuart (51) indicates that in the diametral resilient modulus test the test load becomes a creep load before the deformations are recorded (Figure 5-7). As such, the loading during the resilient modulus tests may not simulate the loads applied by traffic. This factor however is ignored since the magnitude of this discrepancy is too small to cause significant variations in the calculated resilient moduli values in comparison with the other factors.

The equations for the material response given in the previous section were developed based on the assumption that the material is homogenous, isotropic and linearly elastic. Asphalt mixes are non - homogenous and it is doubtful that an asphalt mix would be isotropic if the compaction effects (hence orientation) of the aggregates are considered. However, the assumption of an elastic response is reasonable if the tests are conducted in the linear visco-elastic range using a loading rate which produces low permanent deformations.

When using equation 5-3 to determine the diametral resilient modulus, researchers normally assume a value of 0.35 for the Poisson"s ratio of the mix. Poisson"s ratio is dependent on the binder properties, mix composition, test frequency and the test temperature. However, the effect of above factors has not been firmly established. Small changes in the assumed Poisson"s ratio have little practical effect on the modulus. Ratios between 0.3 to 0.4 are generally assumed when determining the resilient modulus although the values can vary between 0.2 to 0.5. Decreasing the assumed value of the ratio from 0.35 to 0.3 decreases the modulus by 8 percent and a similar decrease from 0.35 to 0.2 results in a 24 percent reduction in the modulus values. Even if it were possible to measure the horizontal and the vertical deformations, the use of these deformations to calculate Poisson"s ratio would still be questionable due to the instruments measurement limitations.

5.3.4 Resilient Modulus Tests on CRM Mixes

In this study, the resilient characteristics of the CRM mixes were determined using the Retsina Apparatus. Prior to testing, three diametrical axes were marked on each specimen and height of the sample was determined on these three axes. The specimens were conditioned for 24 hours in an environmental chamber at the specified test temperature prior to testing.

The sample was placed in the yoke and the four screws were tightened such that that one diametrical axis of the specimen was aligned parallel to the horizontal axis of the yoke. The entire unit was then placed at the center of the loading frame. A steel curved loading strip and a steel ball were used to secure contact with the load cell and the loading frame.

The seating load was set to 22.2 N and the transducer screws were tightened to bring the transducers in contact with the specimen. Upon contact with the specimens, the transducers begin measuring creep deformation of the specimen under the seating load. The dynamic loading and testing was not started until the transducer readings indicated that the creep deformation had ceased. Prior to testing, the LVDT"s were zeroed. Then the dynamic load was applied through the pneumatic unit. The initial dynamic load was recorded at the start of the experiment and six consecutive deformations were recorded. The testing was stopped after recording the final dynamic load.

With a break of about 6 hours, the testing was repeated on one of the other two perpendicular axes. The average resilient modulus values from the three tests was reported as the representative moduli. In this study, the resilient modulus tests were initially conducted at a test temperature of 25 C. There were no definite trends about the benefits from incorporating CRM into the mixes either by the dry (RUMAC mixes) or the wet (A-R mixes) process. In addition, the PG classification of A-R blends did not differ in their intermediate temperature properties tied to the load associated fatigue cracking (Table 3-3). Hence to determine whether or not the use of rubber influenced temperature effects on the resilient modulus it was decided to evaluate the mixes at two additional temperatures of 5C and 25 C. To conduct the tests at 5 C , the loading frame of the Retsina Apparatus was placed in a freezer and the tests were conducted without any difficulty. Table 5-5 shows the testing matrix adopted for the resilient modulus tests.

Table 5-5 Testing Matrix for Resilient Modulus Testing

Mix Type	**Marshall Mixes**			**Superpave Mixes**		
	5C	25C	40C	5C	25C	40C
Unmodified	6	6	6	6	6	6
RUMAC 1%	6	6	6	6	6	6
RUMAC 2 %	6	6	6	6	6	6
RUMAC 3%	6	6	6	6	6	6
A-R 5% CRM	6	6	6	6	6	6
A-R 10% CRM	6	6	6	6	6	6
A-R 15% CRM	6	6	6	6	6	6

Total Number of Resilient Modulus Tests Conducted: 189

5.3.5 Analysis of Resilient Modulus Data

Resilient modulus of the mixes was determined using the diametral test. The modulus was calculated using the equation 5-4 with Poisson"s ratio assumed to be 0.35. Figures 5-8 to 5-11 show the variation of resilient modulus of the mixes (both Marshall and Superpave mixes) with test temperatures.

To analyze the test data and make statistically relevant conclusions about the effect of CRM on the resilient modulus of the CRM mixes, a two factor Analysis of Variance (ANOVA) test was performed using the Statistical Analysis Software (SAS) package (50). The factors considered in this analysis were mix type (4 mix types) and test temperature (three test temperatures). The two factor ANOVA test indicated whether the mix type and temperature had a significant effect on the resilient modulus of the Unmodified & RUMAC mixes, and Unmodified & A-R mixes.

A SAS program written for this purpose provided information in three stages. The first model evaluated the effect of mix type on the resilient modulus, the second model evaluated the effect of test temperature on the resilient modulus, and the third evaluated whether the mix type and test temperature interaction had a significant effect on the resilient modulus. The results from the two factor ANOVA was expressed in terms of the probability (Pr > F) that the factors tested have a significant effect on the resilient modulus of the unmodified and the RUMAC mixes (or the Unmodified and A-R mixes). Probability values greater than 5% indicated that the rutting resistance of the mixes did not differ significantly. For the statistical analysis, the effect of mix and test temperature on resilient modulus were evaluated. The output from the two factor ANOVA was utilized to determine the Least Square Difference (LSD) in the mean resilient modulus at a given test temperature. Using the LSD it was possible to identify the mixes (with in a given set and at a given test temperature) which did show significant difference between the resilient moduli (50). Appendix B shows the SAS program written for two factor ANOVA test along with a sample output.

5.3.6 Effect of CRM on Resilient Modulus

The resilient modulus test results resulted in the following observations:

1. From Table 5-6 it can be seen that the two factor ANOVA test indicates a significant interaction effect of mix type and test temperature on the resilient modulus. In other words, the results can be interpreted as the resilient modulus differences of various mixes are not the same at all temperatures.

Table 5-6 Summary of Two Factor ANOVA Test on Resilient Modulus Test Results

Mix Combination	Probability for Two Factor ANOVA Test	Remarks
	MIX*TEMP	
Unmodified and RUMAC Mixes - Marshall Design	0.0001[a]	Interaction effect of MIX & TEMP Significant on Resilient Modulus
Unmodified and A-R Mixes Marshall Design	0.0001	Interaction effect of MIX & TEMP Significant on Resilient Modulus
Unmodified and RUMAC Mixes - Volumetric Mix Design	0.0001	Interaction effect of MIX & TEMP Significant on Resilient Modulus
Unmodified and A-R Mixes Volumetric Mix Design	0.0001	Interaction effect of MIX & TEMP Significant on Resilient Modulus

[a]Probability value greater than 0.05 is an indication that the effect of mix and temperature is not significant on the resilient modulus

2. Table 5-7 shows the statistical analysis of the resilient modulus test results of the Marshall mixes. Although the differences are not significant from statistical considerations, it can be seen the Marshall - CRM mixes with 1% (RUMAC mix) and

5% (A-R mix) in most cases showed higher resilient modulus when compared to the Marshall -Unmodified mixes. This trend was generally true at all the three test temperatures at which the unmodified and CRM mixes were evaluated in this study. However, increase in CRM content beyond 1% (for RUMAC mixes) and 5% (for A-R mixes), reduced the resilient modulus of the CRM mixes.

Table 5-7 Least Significant Difference (LSD) in Mean Resilient Modulus of the Marshall Mixes Evaluated in this Study

Mix Type	**OAC (%)**	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**
		5 C	**25 C**	**40 C**
Unmod	**5.1**	10.45[e]	1.89[c]	1.02[a]
RUMAC 1%	**5.1**	11.10[e]	1.92[c]	1.24[a]
RUMAC 2%	**5.1**	7.41[d]	4.4[b]	0.56[a]
RUMAC 3%	5.7	7.98[d]	1.19[c]	0.55[a]
LSD (MPa)	***1.94***			
		Mean Modulus (MPa)	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**
Mix Type		**5 C**	**25 C**	**40 C**
Unmod	5.1	10.45[j]	1.89[g]	1.02[f]
A-R 5%	5.2	10.70[j]	3.23[h]	1.00[f]
A-R 10%	5.6	9.70[I]	2.95[h]	0.81[f]
A-R 15%	5.8	9.51[I]	1.61[g]	0.83[f]
LSD (*MPa*)	***0.54***			

Means in the same set followed by the same letter are not significantly different at a = 0.05

3. At 40C, even though the resilient modulus of the CRM mixes decreased with an increase in CRM content in the mixes, the differences in the resilient modulus of the unmodified and CRM modified mixes were not significantly different.

4. From Table 5-8, it can be seen that the incorporation of CRM by both dry and wet process did not enhance the resilient properties of the Superpave - CRM mixes at any of the three test temperatures.

To summarize the findings from the resilient modulus testing program, the use of CRM in very small percentages (1% for RUMAC, & 5% for A-R mixes) improved the resilient characteristics of the resulting RUMAC and A-R mixes, although the improvement was not significant from statistical considerations. However, at higher percentage composition of CRM in asphalt mixes, the resilient modulus of the mixes was significantly lesser when compared to the unmodified mixes.

Table 5-8 Least Square Differences(LSD) in Mean Modulus of the Superpave Mixes Evaluated in this Study

Mix Type	**OAC (%)**	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**
		5 C	**25 C**	**40 C**
Unmod	4.1	15.27[i]	3.64[f]	2.35[c]
RUMAC 1%	4.1	11.82[h]	2.44[e]	1.55[b]
RUMAC 2%	4.1	5.05[g]	1.83[d]	0.71[a]
RUMAC 3%	4.7	6.16[g]	1.62[d]	0.31[a]
LSD (MPa)	***0.45***			
	OAC (%)	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**	**Mean Modulus (MPa)**
Mix Type		**5 C**	**25 C**	**40 C**
Unmod	4.1	15.27[E]	3.64[B]	2.35[A]
A-R 5%	4.4	13.68[D,E]	3.16[B]	2.96[A]
A-R 10%	4.7	12.25[C,D]	3.27[B]	2.49[A]
A-R 15%	4.7	10.80[C]	3.14[B]	1.95[A]
LSD (*MPa)*	***1.65***			

Means in the same set followed by the same letter are not significantly different at a = 0.05

5.4 EVALUATION OF INDIRECT TENSILE STRENGTH OF CRM MIXES

The rutting resistance test program measured the resistance of the mixes to permanent deformation under vertical compressive stresses and while the resilient modulus testing program evaluated the ability of the mixes to bounce back upon releasing the stresses applied on the diametral axis of the asphalt concrete specimens. In this section, the Indirect Tensile Strength testing program will evaluate the tensile strengths of the mixes when subjected to constant strain rate.

The indirect tensile strength test involves loading a cylindrical specimen with either static or repeated compressive loads which act parallel to and along the vertical diametral plane as shown in Figure 5-12. To distribute the load and maintain a constant area, the compressive load is applied through a half-inch wide steel loading strip which is curved at the interface to fit the specimen. The loading configuration develops a relatively uniform tensile stress perpendicular to the plane of the applied load and along the vertical diametral plane which causes the specimen to eventually fail by splitting or rupturing along the vertical diameter (55). The failure mode in a typical indirect tensile strength test is shown in Figure 5-13.

The height and diameter of the samples were determined prior to conducting the test. The samples were conditioned at 25 C for 24 hours in a water bath prior to testing. For testing, the sample was first placed on the lower segment of the breaking head and after placing the upper head, the entire unit was placed under the loading head the MTS Machine. The MTS was set in the "STROKE" mode to cause a vertical movement of 50.8 mm/min. The data acquisition system was set to record the data at 1 second interval and terminate the test at the instant the load begins to decrease.

The maximum load was recorded for each specimen using the "Hold at Break-Point" mode. The Indirect Tensile Strength of the specimens was calculated using the formula

$$ITS = \frac{2000P_{Max}}{?Dt} \quad ....5\text{-}6$$

Where,

ITS = Indirect Tensile Strength (MPa)
P_{max} = Peak Tensile Load (KN)
D = Diameter of the sample (mm)
t = Thickness of the sample (mm)

Table 5-9 shows the testing matrix for Indirect Tensile Strength Tests

Table 5-9 Testing Matrix for Indirect Tensile Strength Tests

Mix Type	**Marshall Design**	**Superpave Design**
Unmodified	3	3
RUMAC 1%	3	3
RUMAC 2%	3	3
RUMAC 3%	3	3
A-R 5% CRM	3	3
A-R 10% CRM	3	3
A-R 15% CRM	3	3

Total Number of Samples for Indirect Tensile Strength Test = 63

To analyze the test data and make statistically relevant conclusions about the Indirect Tensile Strength (ITS) of the CRM mixes, a one factor Analysis of Variance (ANOVA) test was performed using the Statistical Analysis Software (SAS) package (50). This one factor ANOVA test indicated the role of mix type on the ITS of the Unmodified and RUMAC mixes, and Unmodified vs. A-R mixes. The results from the ANOVA test was utilized to determine whether the mix type had a significant effect on the indirect tensile strength of the mixes. The results from the ANOVA test was expressed in terms of the probability (Pr > F) that the effect of mix type on ITS of the unmodified and the RUMAC mixes (or the Unmodified and A-R mixes) being significant. Probability values greater than 5% indicated that the rutting resistance (permanent strain) of the mixes did not differ significantly. The statistical analysis was further extended to determine the Least Significant Difference (LSD) in the mean ITS of a pair of mixes (Equation 5-1) . Any two mixes (from a given set) having a difference in ITS less than the LSD are considered not significantly different Table 5-10 shows the results from the one factor ANOVA test. Appendix C shows the SAS Program for one factor ANOVA and a sample output of the results.

5.4.1 Effect of CRM on Indirect Tensile Strength Properties

From Table 5-10 it can be seen that mix type has a significant effect on the ITS. The Mix types was evaluated in two groups viz., Unmodified mix and the RUMAC Mixes, and Unmodified and the A-R mixes.

Table 5-10 Summary of One Factor ANOVA Test on ITS Test Results

	Probability for One Variable ANOVA Test	Remarks
Mix Combination	**MIX**	
Unmodified and RUMAC Mixes - Marshall Design	0.0001[a]	Effect of MIX is Significant on Tensile Strength
Unmodified and A-R Mixes Marshall Design	0.0002	Effect of MIX is Significant on Tensile Strength
Unmodified and RUMAC Mixes - Volumetric Mix Design	0.0001	Effect of MIX is Significant on Tensile Strength
Unmodified and A-R Mixes Volumetric Mix Design	0.1102	Effect of MIX not Significant on Tensile Strength

[a]Probability greater than 0.05 is an indication that the effect of mix and temperature is not significant on ITS

Table 5-11 shows the Least Significant Difference between the mean ITS values of any two mix within a given set. Table 5-11 indicates that among the Marshall - RUMAC mixes, there is a significant difference between the tensile strengths of the RUMAC mixes and that the incorporation of CRM into the asphalt mixes by dry process reduced the tensile strength of the resulting RUMAC mixes. Similar trends are evident for the Superpave - RUMAC mixes although the tensile strength of RUMAC 2% and RUMAC 3% mixes do not differ significantly.

In case of A-R mixes, it can be seen from Table 5-11 that although the Marshall A-R mixes show higher tensile strengths than the unmodified mixes, the differences is not significant at 5% level of significance. Similarly, the ITS of the A-R and the Unmodified mixes designed by the Superpave method did not differ significantly at 5% level of significance.

Table 5-11 Least Significant Differences (LSD) in Mean Tensile Strength of the Mixes Evaluated in this Study

Mix Type	**OAC (%)**	**Marshall Mixes**	**Mix Type**	**OAC (%)**	**SUPERPAVE Mixes**
		Mean ITS (MPa)	**Mix Type**		Mean ITS (MPa)
Unmod	5.1	1.46[a]	**Unmod**	4.1	1.60[f]
RUMAC 1%	5.1	1.50[a]	**RUMAC 1%**	4.1	1.43[f]
RUMAC 2%	5.1	1.23[b]	**RUMAC 2%**	4.1	1.00[g]
RUMAC 3%	5.7	0.98[c]	**RUMAC 3%**	4.4	0.93[g]
LSD (MPa)		***0.071***	***LSD (MPa)***		***0.160***
Mix Type		**Marshall Mixes**	**Mix Type**		**SUPERPAVE Mixes**
Unmod	5.1	1.46[d]	**Unmod**	4.1	1.60[h]
A-R 5%	5.2	1.96[e]	**A-R 5%**	4.4	1.73[h]
A-R 10%	5.6	1.86[e]	**A-R 10%**	4.7	1.63[h]
A-R 15%	5.8	1.83[e]	**A-R 15%**	4.7	1.53[h]
LSD (MPa)		***0.143***	***LSD (MPa)***		***0.160***

Means in the same set followed by the same letter are not significantly different at a = 0.05

To summarize the analysis of the tensile strength test results of the unmodified and CRM modified mixes, it can be concluded that there were so significant improvements to the tensile strength of the asphalt mixes modified with the CRM either by the dry or the wet process.

5.5 EVALUATION OF FATIGUE CHARACTERISTICS OF CRM MIXES

Fatigue is a flexible pavement associated distress which manifests itself in the form of cracking from repeated traffic load applications. Numerous research projects have been conducted in the past to characterize the fatigue behavior of asphalt mixes. These studies have characterized the fatigue properties of the mixes by relating the initial stress or strain in the mix to the number of load applications to failure (56,57). The fatigue behavior of the mixes have been characterized by the slope and relative level of the stress or strain versus the number of load repetitions to failure. Mathematically, the relationship is expressed as :

$$N_f = a\,(1/e_0)^b\ (1/E_0)^c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ 5\text{-}7$$

where,

N_f = fatigue life
e_0 = initial tensile strain
E_0 = initial mix stiffness
a,b,c = experimentally determined coefficients

An understanding of the fatigue characteristics of the asphalt-concrete mixes over a range of traffic and environmental conditions is essential to incorporate fatigue considerations into the flexible pavement design procedures (56,57). In this study, the RUMAC mixes obtained from the field project were evaluated for their fatigue characteristics by determining the number of load repetitions a test beam of RUMAC mix can withstand under repeated application of bending stresses. The fatigue lives of the mixes were then compared to ascertain if the incorporation of CRM had any significant role in enhancing the fatigue characteristics of the asphalt mixes .

5.5.1 Terminology Associated with the Fatigue Behavior of Flexible Pavements

Fatigue: Repeated application of traffic results in the pavement layers being subjected to varying degrees of stresses and strains. Figure 5-14 illustrates the fluctuating stresses and strains in an asphalt concrete pavement subjected to moving single-axle and tandem-axle loads (56). In this context, Yoder (54) defines fatigue as the phenomena of repetitive load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material.

Navarro and Kennedy (55) quote the ASTM definition of fatigue as a process of progressive localized permanent structural change occurring in a material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in cracks or complete fracture after a sufficient number of fluctuations.

Fatigue in flexible pavements results in the development of alligator cracks in the wheel paths due to the excessive tensile strain at the bottom of the asphalt layer. The fatigue cracking generally originate from the bottom of the asphalt layer (for sections having the granular base) and propagate upwards. This has been confirmed (45) through studies at the Turner Fairbanks Research Center wherein high deflections were measured before cracking appeared on the surface indicating the development of cracks below the surface.

Fatigue Life, Fracture Life and Service life: Fatigue Life (N_f) is normally referred to as the total number of load applications necessary to cause a 50% reduction in the stiffness of the test specimen. Fracture life is the number of load applications required to cause the complete fracture of the specimen. Service Life is the total number of load applications necessary for the test specimen to no longer perform as it was originally intended (56). Figure 5-15 shows the possible definitions of failure of a specimen subjected to laboratory fatigue testing.

Controlled-Stress and Controlled-Strain Fatigue Tests: Fatigue testing is normally conducted by either controlling the load (stress) or the deformation (strain). In the stress-controlled tests, the nominal load, or stress, is kept constant and applied repeatedly until failure occurs. With this type of test, the strain gradually increases as the number of load repetitions accumulate. In strain-controlled tests, the nominal deflection or strain resulting from each load application is kept constant until failure. As the specimen "weakens" the stress required to produce the strain gradually decreases. Table 5-12 reproduced from Rao Tangella Et. al (58), gives a comparative evaluation of controlled-stress and controlled-strain loading

Mode Factor: This is a non-dimensional factor developed by Monismith and Deacon(58) to differentiate between the controlled-stress and controlled-strain tests on a quantitative basis. The mode factor is given by the equation;

$$MF = \frac{/A/ - /B/}{/A/ + /B/} \qquad \text{.......5-8}$$

where,

MF = Mode Factor
/A/ = Percentage change in the stress due to an arbitrarily fixed reduction in stiffness
/B/ = Percentage change in the strain due to an arbitrarily fixed reduction in stiffness

For controlled stress conditions, the change in stress (/A/) is zero while the change in strain for controlled strain conditions (/B/) is zero. Hence the Mode Factor for controlled stress and controlled strain conditions are -1 and + 1 respectively. Based on the elastic layered analyses, Monismith and Deacon (58) concluded that the controlled stress loading is suitable for thin flexible pavements (thickness 50 mm or less) which indicate a mode factor of -1 and the controlled strain loading is suitable for thick pavements (150 mm or greater) which show a mode factor of +1. Figure 5-16 (58) shows the fatigue behavior of asphalt paving materials for various modes of loading.

Simple and Compound Loading for Fatigue Tests: Loading Condition refers to a given set of load and environmental variables adopted for the conduct of fatigue tests. Rao Tangella et al. (58) indicate that a test specimen can be subjected to ***simple loading*** by maintaining constant loading conditions during the fatigue test. However, in actual practice, the pavements are subjected to c***ompound loading*** due to the variations in the traffic-induced loads and environmental conditions. Compound loading can be simulated in the laboratory by a ***sequence repeated block or random tests.*** For ***sequence tests,*** different numbers of load applications N_1, N_2, N_3 are applied at different levels of stresses s_1, s_2, s_3 respectively, until failure occurs; for *r**epeated block tests*** a block of load Figure 5-16 Fatigue Behavior of Asphalt Paving Materials for Various Modes of Loading[58] applications is repeatedly applied until failure occurs; a block is defined as two or more different numbers of applications at different stress levels; and, the block size is the total number of applications within a block. For ***random tests*** the number of applications and the stress level are randomly applied until failure occurs. If the moisture conditions and temperature are varied along with the above mentioned variables, such a test can best simulate the field conditions from traffic and environmental conditions. However, these "Super- Compound" tests are difficult to perform.

5.5.2 Effect of Mix Compaction on Fatigue Characteristics

Rao Tangella et. al (58) indicate that the fatigue response of asphalt pavements are affected by factors like:

1. Specimen fabrication i.e. compaction procedures
2. Mode of loading , environmental conditions and
3. Mixture variables like percent voids, percent asphalt etc.,

Clear understanding of the effect of above variables on fatigue response of mixes aid in developing specifications for mix preparation and specimen fabrication, and help to select the loading and environmental conditions for a fatigue test. Although many sample preparation procedures are available, the criterion for the selection of a fabrication procedure is the ability of the procedure to duplicate the corresponding in-situ asphalt paving from mix composition, density properties, minimum cost, technical skill and time considerations (57). The most commonly adopted compaction methods for sample preparation are static compaction, impact compaction, kneading compaction, gyratory compaction, and rolling-wheel compaction (55,57). Although a detailed discussion on the compaction methods is beyond the scope of this study, Table 5-13 reproduced from Rao Tangella Et. al. (58) gives a relative comparison of the different compaction methods. The researchers of the SHRP project A-003-A rank the rolling wheel, kneading and the gyratory compaction procedures in the order of their ability to produce test specimens which simulate the in-situ mix.

5.5.3 Effect of Mix Variables

The fatigue response of a mix is affected by all those factors that affect the mix stiffness i.e., the asphalt content, viscosity, air voids, temperature and aggregate gradation. Fatigue resistance can be increased by increasing the asphalt content as long as the stability is not affected and by achieving a design density and air voids by adequate compaction. The fatigue resistance of a pavement subjected to heavy traffic can be increased by using a dense graded mix and a stiff asphalt (duly considering the thermal cracking effects). However, the use of asphalt with lower stiffness and softer asphalt are recommended for light-duty pavements (58). The use of rough and angular aggregates is said to increase the stiffness of the mix due to better interlocking.

5.5.4 Effect of Loading and Environmental Variables

The fatigue response of asphalt mixes are affected by the shape and duration of the load pulse and testing temperature. Load duration wave forms that have been used in the fatigue tests are sinusoidal, haversine and cyclic (with various loading time). Figure 5-17 shows the loading patterns adopted in the fatigue tests. The effect of typical wave forms on the fatigue life cycles of a particular mix is shown in Table 5-14. Researchers (58) have studied the effect of equivalent time of loading to the pavement depth and have concluded that a time of loading between 0.04

to 0.1 second is appropriate for fatigue testing. Environmental effects cause an age-induced stiffening of the mix which in turn increases the fatigue life. This stiffening is believed to offset the effect of higher in-situ air voids in the mix and damage due to the traffic. However, the age-induced stiffness can be detrimental to the mix in terms of low temperature cracking due to the increased brittleness (58). Fatigue tests on slabs taken from the in-service pavements have indicated an increase in fatigue life for a given stress level by a factor of 3 and increased dynamic stiffness by 60 percent due to an increase in stiffness and reduction in air voids (58).

5.5.5 Methods of Fatigue Testing

The main objective of a fatigue test is to apply loads to the test specimen which simulate the loading due to traffic so as to induce stresses and strains similar to those produced by the traffic. The environmental conditions during the fatigue test must also simulate the field conditions as closely as possible. Researchers (58) have worked on different fatigue testing methods since 1948 and some of the important fatigue testing methods developed since then are; third point flexural loading, center point flexural loading, cantilever flexural loading, rotating cantilever, uniaxial, diametral, and supported flexural loading. These tests involve a definite loading configuration, wave form and frequencies which create zones of uniform stress. Table 5-15 reproduced from (58) gives an overview of the fatigue tests methods. The detailed description of all the fatigue testing methods is beyond the scope of this study and only those test methods important to the research will be described in the subsequent sections.

5.5.5.1 Simple Flexure Test

In a simple flexure test, a direct relationship is developed between the fatigue life and stress/strain by subjecting the beam specimens to pulsating or sinusoidal (rotating and trapezoidal cantilever beams) loads, (either stress or strain controlled) in a third-point or center-point configuration. Loading continues until the specimens fail or exhibit changes in characteristics which render the mixture unsuitable. The results from these tests take the typical form;

$N_f = a\ (1/s_t)^b$ ....5-10 for stress controlled tests

$N_f = c\ (1/e_t)^d$ ....5-11 for strain controlled tests

where, s_t and e_t are the magnitudes of initial stress and initial tensile strain applied, a,b,c and d are the material coefficient associated with the laboratory test methodology, and N_f is the number of load applications to failure.

Instrumentation for conducting controlled stress or controlled strain fatigue tests with center-point and third-point loading is shown in Figure 5-18. The University of California at Berkeley and the Asphalt Institute use beam specimens of dimensions 37.5 X 37.5X 375 mm and 75 X 75 X 375 mm respectively. The specimens were subjected to pulsating loads with a time of loading of 0.01 sec and a frequency of loading of 100 repetitions per minute. Figure 5-19 is a representation of the typical load and deflection traces.

5.5.5.2 Cantilever Type of Loading

This type of loading has been commonly adopted at the University of Nottingham by Pell et. al., and other researchers (58). In the cantilever type of loading, the test samples are subjected to flexural loads by a rotating cantilever machine (Figure 5-20a) or by sinusoidal loading using trapezoidal beams (Figure 5-21), or controlled-strain torsional testing machine (5-22). For tests conducted under rotating loading, the specimen is mounted vertically on a rotating cantilever shaft, and a load applied at the top to induce a bending stress of constant amplitude through the specimen. The tests are conducted at a test temperature of 10 C and a speed of 1000 rpm. The dynamic stiffness of the sample is measured using another device (Figure 5-20b) which applies constant sinusoidal amplitude deformations. In addition, the cantilever type of loading can also be applied using a controlled-strain torsional testing machine.

5.5.5.3 Diametral Test

Diametral fatigue test is an indirect tensile test conducted by repetitively loading a cylindrical specimen with a compressive load which acts parallel to and along the vertical diametral plane. This loading configuration develops a reasonably uniform tensile stress in along the specimen diameter perpendicular to the direction of the applied load. The test setup used for this test is relatively simple and loads can be applied with devices including electro-hydraulic and pneumatic systems. Researchers (58) have used two different types of loading periods, the first used a loading period of 0.4 second and rest period of 0.6 second, while the second type used a loading period of 0.05 second and a frequency of 20 rpm. For the fatigue tests, haversine load pulse is applied on the test specimens of 100 mm diameter and 62.5 mm height through a 12.5 mm wide loading strip. Rao Tangella et. al (58) indicate that researchers have reported that with a line load of sufficient magnitude, the diametral specimen would fail near the load line due to compression. It is possible to induce tensile failure along the vertical diameter by applying a sufficiently large load and a loading strip to distribute the compressive load over the length of the specimen. Researchers (51) have used the types of failure due to loading on the diametral axis of the specimen to determine whether the failure was predominantly due to tensile strain or not. Figure 5-23 shows the possible ways a cylindrical sample can fail under diametral loading. Figure 5-24 shows the stresses at the center of the specimen due to a strip load applied on the diametral axis. The equations to determine the magnitude of tensile and compressive stresses at the center of the specimen are as follows;

$$s_t = \frac{(2P)}{?\ a\ h} \quad [\ \sin 2a - \{a/(2R)\}] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 5\text{-}12$$

$$s_c = \frac{(-6P)}{?\ a\ h} \quad [\ \sin 2a - \{a/(2R)\}] \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 5\text{-}13$$

where,

P = Applied load
a = Width of the loading strip
h = Height of the specimen
R = Radius of the specimen
2a = Angle at the origin subtended by the width of the loading strip
s_t = Horizontal indirect tensile stress at the center of the specimen
s_c = Vertical indirect compressive stress at the center of the specimen

From the above two equations, it can be seen that the vertical compressive stress at the center of the specimen is three times the horizontal tensile stress.

5.5.6 The Fatigue Testing Program

The evaluation of CRM mixes for fatigue characteristics was the final phase of the research project. In this phase, the CRM mixes were evaluated for their fatigue life by subjecting the beam samples of CRM mixes under cantilever type of loading. The cantilever type of loading resulted in subjecting the beam samples to uniform shear between the fixed end and the loading point, and to a bending moment which varied from zero at the loading point to a maximum value at the fixed end. The fatigue life of the mix was measured in terms of the number of load cycles required to cause a 50 percent reduction in the initial stiffness of the mix under repeated bending. The data pertaining to the initial strain and fatigue life has been used to evaluate the benefits of using CRM as an additive in asphalt mixes.

Slabs of size 600 X 300 X 75 mm were first sawed from the experimental stretches (with 1, 1.5 and 2% CRM overlays) on Interstate 40 near Russellville, Arkansas, using a high speed diamond saw. The slabs were subsequently removed from the pavement by using a jack hammer. The slabs were then trimmed along the sides and further sawed in the laboratory to beams having dimensions of 275 X 72.5 X 72.5 mm. These dimensions were used so as to obtain the maximum number of beam samples for each slab.

The beams were tested for Bulk Specific Gravity and the Theoretical Maximum Density

(TMD) of each mix type was determined using the left over chunks from the slabs. Using the BSG and TMD the volumetric properties of each beam sample were determined.

5.5.6.1 Selection of Fatigue Testing Method

The fatigue testing of asphalt mixes using beam samples was being attempted for the first time at the University of Arkansas through this research program. Since the research staff had no prior experiences with the development of a fatigue testing unit, literature review was first conducted to understand the principles behind the fatigue testing procedures and to identify a test fixture that could be developed with minimal time and resources. Initially, a simply supported beam with third point loading was selected for the fatigue testing based on its apparent simplicity. However, several problems were encountered that resulted in the abandonment of this test approach.

The first problem was with the loading system used to apply the two-point loading. Initially, the loading head with two rollers was placed directly on the sample and the load was applied on the loading head through a piston attached to the MTS. The

weight of the loading head posed problems in terms of applying dead load to the beam that caused the specimens to fail without the test load being applied if the loading was delayed too long. To eliminate this problem, the loading head was attached to the piston to act as weight of the loading head posed problems in terms applying a dead load to the beam that a component of the loading piston. This eliminated the application of dead load on the beam samples and minimized the errors in fatigue testing. Figure 5-25 shows the two arrangements.

The second problem was that the simple two-point load application on the asphalt beams failed to simulate the fatigue loading. This came into focus during the data analysis. From Figure 5-26 it can be seen that the drop in the stiffness ratio from 1.0 to 0.5 over a 3000 load repetitions indicates excessive permanent strain undergone by the beam under the third point loading, which may be more indicative of rutting potential of the mix rather than the fatigue resistance.

To overcome this problem, a new accessory was fabricated to hold the specimen at the ends such that the load application would result in flexing of the beam to a predetermined amount on either side (up and down) of the horizontal neutral axis of the beam. This arrangement of flexing the beam by a predetermined amount permitted the test to be conducted under the strain control mode without difficulty. The maximum tensile strain that developed at the bottom most fiber of the beam (under the controlled strain conditions of testing) was determined at the region of the maximum bending moment (midway between the two loading points) using a strain gauge. Figures 5-27 and 5-28 show the original and modified setup.

Although the new setup overcame most of the previously encountered problems, beam samples tested using this arrangement did not fail in the zone of maximum bending moment, i.e., between the two loading points. The samples instead failed under the loading points. Also, the wing nuts loosened during testing which caused excessive vibration of the beam testing unit during the load application cycles. To prevent this the wing nuts were tightened over the heavy duty springs having a load carrying capacity of about 100 kg. Figure 5-28 shows the beam fatigue test set up with accessories to hold the beam and the heavy duty springs inserted to prevent the vibrations. Although the use of heavy duty springs alleviated this problem, the beam failure still occurred at the loading points and the end supports rather than in the region of maximum bending moment.

Shortage of samples forced the consideration of a beam flexure testing method which involved minimum number of variables in the instrumentation. A cantilever type of loading was selected for applying the flexure load on the beam. A new fixture, in which the beam sample is fixed at one end and the load applied at the free end was fabricated. This fixture permitted the application of a load of sufficient magnitude to cause an equal amount of movement on either side of the horizontal neutral axis. Figures 5-29 shows the concept of cantilever loading for the fatigue testing and Figure 5-30 is a simple line sketch of the cantilever loading unit for fatigue testing.

The cantilever type of loading adopted in this research program does not conform to the SHRP specifications for evaluating the fatigue characteristics of asphalt concrete beams. The two point loading for the beam tests was selected by the SHRP researchers because of the researchers" familiarity, sophistication of its current design, and software interface. But the SHRP Researchers (57) considered the beam and cantilever tests as equivalent means of assessing the fatigue behavior of asphalt-aggregate mixes even though the two test methods have their limitations in terms of the inability of the beam testing to reasonably demonstrate the effect of asphalt content on cycles to failure, and the questionable stiffness-temperature effects of the mixes when tested under cantilever loading.

In this study every attempt was made to develop a fatigue testing system that could provide results consistent with the SHRP fatigue testing units. The fatigue testing program was a relatively small portion of the overall study. As such the resources were not sufficient to develop a full fledged fatigue testing unit. A fatigue test method, based on sound principle of the statics and capable of applying bending stresses to the asphalt mixes had to be developed for this study to obtain information about the benefits of incorporating CRM into the asphalt mixes.

The cantilever type of loading finally satisfied the requirements and was chosen for evaluating the CRM mixes. This instrumentation was capable of subjecting a beam sample to bending and produce reproducible results. Although there exists a tremendous scope to improvise the instrumentation, it was beyond the scope of the research project to venture into this side study.

5.5.6.2 Description of Cantilever Type of Loading System for Fatigue Tests

The basic premise behind the cantilever type of loading system was to subject the free end of the cantilever beam (of CRM mix) to a sinusoidal loading to cause a predetermined amount of displacement on either side of the horizontal neutral axis. The repeated application of the bending stresses on the beam caused a reduction in the beam stiffness. The number of load cycles required to cause a 50 percent reduction in the mix"s initial stiffness was considered as the fatigue life of the mix under consideration.

The test set up shown in Figure 5-30 essentially consists of 1) a fixture for holding the specimen and 2) a loading frame to apply the bending stresses on the beam. The fixture holds the beam rigidly and provides the fixed support of the cantilever beam. A loading head attached to the MTS machine through the load cell rests on the free end of the beam. The loading head is clamped to the free end of the cantilever beam such that, when a sinusoidal loading is applied through the MTS, the cantilever beam is subjected to a predetermined amount of displacement (flexing) on either side of the horizontal neutral axis.

To ensure that the loading does not cause stress concentrations at either the fixed or under the loading position at the free end, the edges at those position were rounded and leather strips were placed under the loading position. In addition, heavy duty springs were used to prevent the loosening of the bolts.

Two fixtures of identical dimensions (shown in Figure 5-31) were used to measure the free end deflection to which the beams were subjected during the fatigue tests. One of them was glued to the free end of the cantilever beam while the other was fixed to the MTS platform. The strain gauge was attached to the free ends of the two fixtures. This setup permitted the measurement of the free end deflection of the beam during testing.

5.5.6.3 Preparation of the Test Specimen for the Fatigue Tests

To prepare the beams for fatigue testing, the beams were conditioned for at least 24 hours at 25 C. After recording the beam dimensions a fixture was glued to one of the ends of the beam to facilitate the attachment of a strain gauge for measuring the free end deflection of the fixed beam. The fixture was glued such that its horizontal axis was 37.5 mm (1.5 inches) from the base of the beam.

The fixture was loosened to accommodate the beam sample between two parallel plates (Figure 5-30). In this position the second identical fixture was glued on to the MTS platform to set the beam span to 225 mm (9 inches). The glued position of the fixture was left undisturbed throughout the testing program to maintain a span of 225 mm.

After securing the beam rigidly between the parallel plates and setting the beam span to 225 mm, the loading head was moved down very cautiously to make minimal contact with the beam. In this position, the free end of the beam was attached to the loading head using threaded rods and wing nuts. This arrangement permitted the loading head to hold the sample and apply the displacement on either side of the beams" horizontal neutral axis. The beam was now ready for testing. At this stage, it must be noted that leather strips were placed under the loading position prior to clamping the beam to eliminate stress concentration under the loading head. Also, during the test setup the beam was supported sufficiently to prevent sagging of the beams in freely supported condition (i.e., prior to clamping).

5.5.6.4 Parameters Adopted for the Fatigue Tests

The parameters adopted in this study were: beam span of 225 mm (9 inches), loading frequency of 5 Hz i.e., 5 cycles/sec, free end deflection levels of 0.127, 0.195 and 0.254 mm on either side of neutral axis, and a test temperature of 25 C.

With reference to Figure 5-29, the initial mix stiffness was determined by utilizing the equation to determine the free end deflection in the beam. The bending theory principles was applied to determine the initial bending stress in the beam. The initial tensile strain was calculated using the initial bending stress and the initial mix stiffness. The steps involved in the determination of the initial tensile strain from the free end deflection of the beam are given below:

Free end deflection (?) = [Pa^2/ 6IE] [3L-a] 5-14

Stiffness (E) = Pa^2 [3L-a] / 6I?...................5-15

Where,

? = Free end deflection in the cantilever beam due to load P (Figure 5-29)
P = Load applied on the beam
a = Distance between the loading position and the fixed end (125 mm)
L = Beam span (225 mm)
I = Moment of inertia [$BD^3/12$]
B = Width of the beam (about 75 mm)
D = depth of the beam (about 75 mm)
E = Stiffness of the beam (Figure 5-29)

The tensile stress at the top fiber of the beam is determined using the principles of Universal Bending Theory.

Tensile Stress f = M y/I 5-16

Where,

M= Maximum bending moment (Pa) due to load P (Figure 5-29)
f = Tensile stress in the beam due to the load P (i.e., Pa/BD^2)
y = depth to the neutral axis (D/2)

Since the stiffness of the beam in tension and compression are equal as per the assumptions of the Bending Theory, the tensile strain at the top most fiber of the beam (at the fixed end) due to a load P can be determined as;

Tensile Strain $e = f / E$ ....""""""" .5-17

Tensile Strain $e = [3D?_{max} / a(3L-a)]$ "" .5-18

5.5.6.5 Fatigue Test Procedure

After clamping the beam to obtain a fixed end condition at the support, the testing system was interfaced with the data acquisition system. At every 120 seconds during the testing process, the following data were recorded; load cycles, deformation and the load applied to the test beam. The strain readings were zeroed using the strain control mode on the MTS machine and the MTS settings were adjusted to cause a targeted free end displacement on either side of the horizontal neutral axis of the beam. The repeated application of free end displacement resulted in the bending stresses on the beam.

The repeated bending stresses on the beam reduced the beam stiffness which caused the initiation of fatigue cracks at the region of maximum bending moment, i.e., at the fixed support. As the crack propagation continued, the stiffness of the mix reduced thereby reducing the magnitude of the load required to maintain the strain level. The testing was continued until the magnitude of the load dropped to about 25 percent of the initial load (set at the beginning of the test). At this stage, the beams had almost completely cracked clearly indicating that they could not take any additional loads.

After the testing was terminated, the data was saved, the program was terminated, the hydraulic pressure was turned off, and the failed beam sample was removed from the testing unit. The test procedure was repeated for other beams to evaluate the fatigue characteristics of the CRM mixes at the free end deflection levels of 0.127, 0.190 and 0.254 mm respectively. Figure 5-32 shows a typical graph which shows the variation of load and deformation levels during the fatigue tests. In this study, the fatigue tests were monitored in terms of free-end deflection levels because the measurement of free end deflection was easier when compared to the measurement of the tensile strain in the beam sample at the top fiber. Each free-end deflection level corresponded to a definite magnitude of tensile strain at the top fiber of the cantilever beam. The tensile strain was calculated using the beam dimensions, amount of free-end deflection and the magnitude of load applied during a given load application. For the beam dimensions adopted in this study (span 225 mm, "a" 125 mm, beam depth 75 mm, and beam width 75 mm), the free-end deflection levels of 0.127, 0.195 and 0.254 mm correspond to tensile strains of magnitude 4.15, 6.21 and 8.31 X 10E-4 mm/mm respectively.

5.5.6.6 Analysis of Fatigue Test Data

The data acquired during the fatigue test was loaded into a MS Excel worksheet. Calculations were made to determine the mix stiffness at all load cycles using the beam dimensions, load and the deformation data. The mix stiffness and the tensile strain level at 600th load repetition (first data point) was selected as the initial stiffness and initial strain level for analysis purposes. The stiffness ratio was determined at each load repetition as the ratio of the mix stiffness at a given load cycle to the initial stiffness ($E_{ith\ cycle}/E_{initial}$).

The fatigue life of the mix was defined as the number of load cycles (or load repetitions) at which the Stiffness Ratio reduced from 1.0 to 0.5. Figure 5-33 shows the typical variation of Stiffness Ratio with the load cycles for the mixes evaluated in this study.

The test results were first compiled to check the reproducibility in the test results. Subsequently test data were further utilized to plot the variation of the fatigue lives of the RUMAC mixes with the initial tensile strain level and generate prediction equations between the initial tensile strain and fatigue life of the mixes.

5.5.6.7 Discussions on the Fatigue Test Results

During the development of a fatigue testing unit for the study, several field samples were utilized to evaluate the working of the third point and the cantilever type of loading system. This resulted in the shortage of field samples during the evaluation of the RUMAC mixes for their fatigue characteristics. Since only five samples of each mix type were available for fatigue testing, it was decided to test two samples at a free-end deflection of 0.127 mm (tensile strain = 4.15E-4 mm/mm) and one sample each at a free-end deflection levels of 0.195 mm (tensile strain = 6.21E-4 mm/mm) and 0.254 mm (tensile strain = 8.35 mm/mm) respectively. The remaining sample was kept for cross checking purposes. This helped in the generation of regression equations to predict the fatigue lives of the CRM mixes from the initial tensile strain in the mix.

For the sample size used in this study, the cantilever type fatigue testing unit was found to produce reproducible results (Table 5-16). The percentage variation between the test results for RUMAC mixes tested at 0.125 mm free end displacement level (tensile strain = 4.15E-4 mm/mm) were 2.2%, 13.2% and 0.11% for the RUMAC 1, 1.5 and 2% CRM mixes respectively. Although the RUMAC 1.5% mixes show higher variability in the test results when compared to the RUMAC 1 and 2% CRM mixes. Due to the small sample size adopted in the fatigue testing program, it was not possible to pin point the causes for the variability to either to the defects in the beam sample or to the instrumentation.

Figure 5-34 shows the variation of fatigue life with the initial tensile strain in the beam specimens. It can be seen that the fatigue life of the CRM mixes decrease with an

Table 5-16 Reproducibility in the Fatigue Test Results

Mix Type	Free end deflection level	Sample Size	Mean	CV%
CRM 1%	0.125	2	624738 605211	2.2
CRM 1.5%	0.125	2	242186 200597	13.2
CRM 2.0%	0.12	2	113557	0.11

			113738	

increase in CRM content and initial tensile strain level respectively. In other words, the incorporation of crumb rubber into the mixes by the "DRY" process did not enhance the fatigue life of the CRM mixes. This trend is similar to the trends that are evident from the rutting, resilient modulus and the tensile strength tests on the RUMAC mixes that have were discussed in Sections 5.2, 5.3 and 5.4.

Considerable objections can be raised for the development of a prediction equation based on testing one to two samples at a given tensile strain rate. However, this was the best and only option available to obtain maximum information about the fatigue characteristics of RUMAC mixes. It should be noted that similar sample sizes (two) were used in the experimental design under the SHRP research program (57).

The prediction equations which indicate an r^2 values close to 1 must be used with caution. It must be realized for RUMAC 1 and 1.5% CRM mixes, the samples were evaluated at only two tensile strain levels and it is obvious that the regression equation will pass through these two data points to yield a regression coefficient of 1. This points out the limitation of the prediction equations that were developed in this fatigue testing program.

It is essential to evaluate the RUMAC mixes at additional tensile strain levels to obtain prediction equations that can be used for mix evaluation purposes. However, the limitation of the prediction equations does not down play the fact that increasing the CRM content decreased the fatigue life of the resulting RUMAC mixes.

The fatigue testing program brought into focus a key limitation of evaluating the field samples to draw conclusions about the fatigue characteristics of RUMAC mixes. The air-void content in the beam samples taken from the pavement were 6% for RUMAC 1 and 1.5% CRM mixes and 9% for the RUMAC 2% mixes (Table 5-17). Since the air-void level is higher than allowed by the AHTD specifications (no greater than 4%), the RUMAC field samples are not acceptable from compaction considerations. This problem was realized during the initial stages of the study. Attempts were made to stretch the resources and fabricate the beam specimens in the lab by compacting loose field mixes in a steel mold using a small roller. Difficulties associated with the achieving of the desired air-void level in the mixes (between 3 to 5%) and the funding constraints forced the research staff to confine the fatigue program to the evaluation of the field beams only.

Table 5-17 Air Void content in RUMAC Mixes Evaluated for Fatigue Characteristics

Mix Type	Design AC%	Bulk Sp Gr.	Th. Max. Density	Air-Voids	CV%
RUMAC 1%	5.1	2.273	2.417	6.0%	0.8%
RUMAC 1.5%	5.6	2.251	2.394	6.0%	0.3%
RUMAC 2.0%	5.7	2.161	2.377	9.1%	2.0%

In summary, the addition of crumb rubber did not enhance the fatigue lives of the RUMAC mixes. This trend is consistent to the trends observed in the rutting, resilient modulus and indirect tensile strength testing programs.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The research discussed in this report investigated into the role of Crumb Rubber Modifiers (CRM) in enhancing the performance properties of asphalt mixes. The entire research was accomplished in three phases, viz., binder evaluation, mix design, and mix performance evaluation. The binder evaluation attempted to characterize the A-R binders in terms of their contribution to increased resistance to rutting, fatigue and thermal cracking. The mix design program evaluated the effect of CRM on the volumetric properties of mixes (prepared by DRY and WET process) designed using the Marshall and Superpave volumetric mix design methods. Performance property evaluation studies evaluated the effect of CRM on rutting, resilient, tensile or fatigue characteristics of the resulting RUMAC and A-R mixes.

The binder evaluation was accomplished using the Superpave binder testing instrumentation, the CRM mixes were designed using the Marshall and Superpave methods, and the rutting, fatigue and indirect tensile strength tests of the mixes were determined using the MTS device with appropriate accessories. The resilient modulus testing was accomplished using the Retsina apparatus with environmental chambers capable of conditioning the mixes from 5 to 40 C. Findings of the three-phase research program are summarized in the following sections.

6.1 RHEOLOGICAL PROPERTIES OF ASPHALT-RUBBER BINDERS

The rheological evaluation of asphalt-cement binder modified with CRM indicated that blending CRM with asphalt increased the low and high temperature range of application of the binder in the field, thus giving an evidence that the AC - CRM interaction can offer potential benefits to the asphalt mixes in terms of increased resistance to thermal cracking and rutting.

6.2 DESIGN OF ASPHALT MIXES MODIFIED WITH CRM

The design of CRM mixes by the Marshall and Superpave Volumetric mix design method indicated that the CRM mixes designed by Superpave method had a lower optimum asphalt content than the CRM mixes designed by Marshall method. The reduction was attributed to the absorption of the asphalt/binder by the aggregates and the CRM during the 4 hour short term aging of the mix - a process which is a true representation of the field aging of the mix from the point of mix production to final laydown and compaction.

6.2.1 Comparison of Mix Designs of RUMAC and A-R Mixes

Incorporation of 1 and 2% CRM into the Marshall mixes did not have any significant effect on the design asphalt content (OAC) indicating the possibility of inadequate reaction between the asphalt cement and CRM particles in the dry process of incorporation of CRM into the asphalt mixes. However, mixes with 3% CRM content showed an significant increase in OAC indicating that an increased absorption of asphalt by the CRM which increases the asphalt content requirements to attain the design volumetric properties.

The wet process A-R mixes which use pre-blended asphalt and CRM blend for A-R mix preparation were less affected by the variation in the OAC when compared to the dry process mixes - thus emphasizing the benefits of using the pre-blended A-R binder to ensure adequate reaction between the asphalt and the CRM particles. The general trend observed from the mix design program is that the RUMAC mixes show a significant reduction in mix stiffness with an increase in CRM content in the mix in terms of Marshall stability.

6.2.2 Significance of Sample Confinement and Mold Paraffining

This side study was undertaken to assess whether paraffining the Marshall molds and sample confinement (prior to extrusion from the Marshall Molds) had a significant effect on the mix design properties of the RUMAC mixes. This study indicated that the mix design parameters of the RUMAC mixes evaluated in this study were not affected by mold paraffining or sample confining procedures.

6.3 PERFORMANCE EVALUATION OF CRM MIXES

The Repeated Load Dynamic Compression Tests conducted at 40 C to evaluate the rutting resistance of the CRM mixes indicated that the incorporation of 1% (RUMAC mixes) and 5% (A-R mixes) CRM into asphalt mixes enhanced the rutting resistance of the resulting RUMAC and A-R mixes, although the improvement was not significant from statistical considerations. CRM content in excess of 1% in RUMAC mixes proved detrimental from rutting considerations. Among the Marshall - A-R mixes, increase in CRM content enhanced the rutting resistance of the resulting A-R mixes as determined using the repeated load dynamic compression tests.

Although the Superpave mixes showed higher rutting resistance (in terms of permanent strain) when compared to the RUMAC mixes, this trend was not considered to be significant because none of the Superpave mixes satisfied the VMA criteria.

The resilient modulus tests conducted on the unmodified and CRM mixes at 5, 25 and 40 C indicated that the incorporation of CRM in excess of 1% (RUMAC) and 5% (A-R mixes) generally decreased the resilient characteristics of the resulting RUMAC and A-R mixes. At 40

C, there was no significant difference between the resilient moduli of the unmodified and RUMAC mixes, and Unmodified and A-R mixes.

It must be recognized that small amounts of CRM (1 and 5%) generally enhanced the resilient modulus of the resulting RUMAC and A-R mixes although the improvements were not significant statistically.

The ITS tests on the unmodified and CRM mixes at 25 C indicated that the Marshall-RUMAC mixes showed a reduction in ITS with an increase in CRM content. The Marshall A-R mixes however showed an improvement in the ITS with an increase in CRM content, an improvement which was significant from statistical considerations.

The fatigue testing program conducted at 25 C using the new fatigue test set up indicated that an increase in the CRM content in the RUMAC mixes reduced the fatigue life. The reduction in the fatigue life was evident at the two initial tensile strain levels at which RUMAC mixes were evaluated.

6.4 LIMITATIONS OF THE FINDINGS FROM THIS STUDY

The materials used and test methods adopted in this study are typical of those currently used by the State of Arkansas. Because the research was limited to a single aggregate blend, crumb rubber and a single asphalt cement type, the findings and conclusions may not be universally applicable. Some of the aspects which limit the universal application of the findings are:

1. In the asphalt-rubber evaluation program, the GF-80 crumb rubber supplied by the Rouse Rubber Industries Inc. was blended with the unmodified AC-30 (supplied by the Lion Oil Company) using a mechanical mixer. No modifiers were used to alter the properties of the blends from viscosity considerations nor there was any measurement of the extent of reaction between the asphalt and the CRM particles during or at the end

of blending period.

Here is must be recognized that the commercial forms of Asphalt-rubber are prepared by blending the materials in presence of undisclosed modifiers to impart specific properties to the A-R blends. The properties of the A-R blends (or the A-R mixes) evaluated in this study may not compare with the properties of the commercial A-R blends or the A-R mixes prepared using these commercial blends.

2. An important factor affecting the performance properties of CRM mixes is the extent to which the CRM particles disperse in the mixes. Segregation of the CRM particles in the mix could affect the performance property trends. In fact some of the inconsistencies in the performance property trends might be tied to the difficulties faced in ensuring uniform dispersion of the CRM particles in the CRM mixes.
3. In this study, the rutting resistance of the mixes were evaluated using the Repeated Load Dynamic Compression Tests. This instrumentation mainly evaluates the resistance of a given mix to permanent deformation under vertical compressive stresses with minimal shearing of the sample. Some researchers (43) claim that shear stresses play an important role in asphalt pavement rutting. This suggests that the rutting resistance of the asphalt mixes evaluated in this study by the repeated load test may not a true measure of the rutting resistance of the mix.
4. A general comment on the statistical analysis used in this study is that the sample sizes for the analysis were not adequate. The sample sizes were two for fatigue tests, three to evaluate the effect of mold paraffining and sample confinement, twelve to evaluate rutting resistance and ITS, and twenty-four to evaluate the effects of CRM on the resilient modulus. It is essential to have large sample sizes to identify whether small differences in the performance properties two mixes (say Unmod and RUMAC 1%) are significant from statistical considerations.

Since the sample size used in this study was small, the comparison between two mixes may provide an inference that difference in performance properties are not statistically significant while from practical considerations they appear to be significant.

5. In this study the availability of the Superpave Gyratory Compaction was utilized to design the CRM mixes using the Superpave volumetric mix design method for a traffic level and environmental conditions typical to the State of Arkansas. The aggregate gradation used for the Superpave mix design satisfied the requirements for the restricted zone but not the control points criteria. The main objective of the designing the mixes by Superpave method was to identify the differences in the mix design parameters of a mix when designed by two mix design processes.

   The mixes designed by Superpave volumetric method did not meet the design criteria but were evaluated for performance properties to observe the trends in the performance properties of the asphalt mixes having varying amounts of CRM in them.

6.5 CONCLUSIONS AND RECOMMENDATIONS

Based on the summary of test results discussed in the previous sections, the following general conclusions were developed relative to the benefits of using CRM in asphalt mixes.

1. The asphalt-rubber blends evaluated in this study showed improvement in the performance properties in terms resistance to rutting, load associated fatigue and thermal cracking. Similar improvements were realized in the Arkansas Type II surface course mixes which were modified with 1% CRM in case of RUMAC mixes and 5% CRM in case of A-R mixes. The improvements were however not significant from statistical considerations.

   CRM content in excess of 1% (RUMAC mixes) and 5% (A-R mixes) was detrimental to the mix performance in terms of rutting, resilient modulus, tensile strength and fatigue characteristics.

2. In light of this finding, there is a need for the asphalt researchers to thoroughly understand the behavior of A-R blends prior to undertaking studies to evaluate the CRM mixes (designed by the conventional methods) for their performance properties. Through a thorough understanding of the behavior of A-R blends (or CRM particles) when mixed with the aggregates, it would be possible to identify the factors that play a significant role in improving the performance properties of the CRM mixes.

   Design of CRM mixes without a thorough understanding of the influence of rubber on asphalt-aggregate interaction will make it difficult to justify the use of CRM in asphalt mixes. It is hoped that further research be directed to address the issues pertaining the asphalt-rubber interactions prior to evaluation of the performance properties of the CRM mixes.

3. This study has put forth a new testing procedure for evaluating the fatigue characteristics of the asphalt mixes. It is essential to perform a ruggeddness testing of this instrumentation to identify those aspects of the instrumentation needing refinements. Some of the refinements that can be recommended to the cantilever fatigue testing unit would be the use of additional bolts to provide a stronger fixed end support to the beam, and a temperature chamber to conduct tests at different test temperatures.

   The fatigue testing program relied solely on the samples obtained by sawing the slabs obtained from the field sections. There is a strong need to develop a methodology for preparing beam samples in the laboratory for fatigue testing. Such a methodology will help in the evaluation of fatigue characteristics of asphalt mixes (both lab and field mixes) designed for various traffic and environmental criteria.

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APPENDIX A

SAS PROGRAM FOR ONE FACTOR ANOVA TEST ON RUTTING RESISTANCE DATA AND SAMPLE OUTPUT